REPORTS, PAPERS, DISCUSSIONS, AND MEMOIRS

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THE MODERN CITY
AND THE ENGINEER'S RELATION TO IT

ADDRESS AT THE ANNUAL CONVENTION AT
CINCINNATI, OHIO, APRIL 22, 1925

By Robert Ridgway,* President, Am. Soc. C. E.

Perhaps the most notable effect of the application of those laws of Nature which have been brought to light by the patient investigations of the scientist during the past century and a half is evidenced in the wonderful growth of cities everywhere, and, as the engineer has contributed so largely to this result, I have chosen to make the theme of my address: "The Modern City and the Engineer's Relation to It," realizing that the thoughts expressed are not original with me, but have long been in the minds of others.

In the simple days when our Federal Constitution was adopted, we were essentially an agricultural people and manufacturing was but an incident. The planter was the man of affairs and when he came to town he was shown the deference befitting his standing in the community. Then, the steam engine began to play its important part in the affairs of the nation and among other applications made transportation over long distances easier. This was followed by the "magnetic telegraph", the result of the good work of Henry and Morse, and, since then, one invention has followed another at an increasing rate up to this time. We do not often stop to think how recent and how new nearly all the inventions which contribute so much to our comfort and well-being really are. It is necessary to go back only to the infancy of those now living to realize this. When I was born, the telegraph was still regarded as a marvel and the locomotive was rather a primitive machine if compared with the great engines of to-day. No trans-Atlantic cable had been successfully laid. The telephone was unthought of and electric lighting and traction were still the dreams of scientists. Water powers were largely going to waste because, although their potential value was understood, the market for the power did not exist and methods of harnessing it for the production of hydro-electric energy were unknown.

The average man, then much more than now, was reluctant to accept these new ideas, which disturbed his habit of thought and action. We are amused when we read of the opposition, violent at times, to the introduction of inventions that later contributed so much to the upbuilding of communities and are now regarded as commonplace. The following extract from "The World To-morrow" is of interest as illuminating this state of mind:

"* * * the following resolution passed in 1828 by the School Board of Lancaster, Ohio, and once quoted by Dr. Fosdick: 'You are welcome to

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* Chf. Engr., Board of Transportation, City of New York, New York, N. Y.

use the school-house to debate all proper questions in. But such things as railroads and telegraphs are impossible, and rank infidelity. There is nothing in the word of God about them. If God had designed that his intelligent creatures should travel at the frightful speed of fifteen miles an hour by steam, He would have foretold it by the mouth of His holy prophets. It is a device of Satan to carry the souls of the faithful down to Hell.'"

Had one possessed a modern radio set in those days and had dared to exhibit it in action, I fear he would have taken his life in his hands. When one of my distinguished predecessors, after his retirement from the office of President, took up in a serious way the problem of æronautics, there were those engineers as well as others, who showed concern for him and wondered whether he had taken leave of his senses. Most people then were content to wait until they had departed from this world before learning to fly. We are amused at these evidences of conservatism, but we must remember that the mass of mankind is naturally conservative by nature, thus keeping the world within bounds; otherwise, it would be taking up every ill-digested idea that was plausibly presented to it. Most men have neither the time nor the qualifications to rightly analyze every new condition as it develops and to differentiate between the few ideas that are good and the many that are worthless. How many fortunes have been lost by investors in patents which were intended by their inventors and claimed by their promoters to revolutionize established methods and which failed to accomplish their purposes because of basic defects?

Modern man is inclined to take as a matter of course all the many developments of scientific research as soon as they become available, and they come at an ever-increasing rate. It is only the thinking man who marvels at the wonderful results which are the outcome of patient study by the scientist and faithful application by the engineer. It is singular how soon men adapt themselves to the changes brought about by new inventions. Most men accept them as they accept sunshine and rain without attempting to understand them. To the average man even the radio has ceased to be a nine-days wonder and an aeroplane barely receives passing notice from him. He knows nothing of the pioneer work of Hertz and has never heard of the contributions to the art of flying made by Langley and Chanute under discouraging conditions before the Wrights made flying an accomplished fact. The pure scientist has been likened to a dreamer interested in a subject or investigation for itself alone without any definite idea of its practical application. He discovers the fact and thus enables the engineer to apply it to a definite and useful purpose. It is extremely rare to find the pure scientist and the applied scientist or engineer combined in one personality. Times without number the former has not received from the world that recognition which was his due. His work in the quiet of the study or the laboratory is not, as a rule, spectacular, and it must be remembered that few scientific truths are discovered by chance.

"Patient research, toil and the effort of years constitute the denominator in the fraction which represents the formula of scientific accomplishment and in this fraction the numerator looms large with dreams of hope, of success, and of future achievement. Without hope, without vision, and without dreams success would ever be beyond attainment."

To myself I have often pictured the world as it might now be if scientific development had begun earlier and had progressed at a less feverish rate than during the past few generations. When, with members of the Board of Direction of the Society, I visited Muscle Shoals on the Tennessee River in 1923, I could not help thinking of what a different slant our history might have taken if the aboriginal red man had been gifted with an understanding of what the energy represented by that falling water meant and had been able to apply that knowledge for the good of his race.

We should not give those of the past century the entire credit for all the wonderful developments which have been made, because others were patiently laying the foundations for them centuries ago and without the good work which they did we would be far behind the point that has now been attained. The first steps in progress tend ever toward extravagance. The marvellous development of our own country during the Nineteenth Century was made at great expense. Our natural resources, such as timber, coal, and wild animal life, appeared to be inexhaustible and shocking waste resulted. Now that these resources have been depleted, and the exhaustion of some of them is in sight, a policy of conservation becomes the order of the day. Europe long since learned the lesson of economy, but we are fast learning the same lesson which was forced on the older countries long before it was realized or heeded here. The world looks to the scientist, to the engineer, and to the chemist for assistance in learning how to make the best use of our remaining natural wealth. Their problem, however, goes further. They must point the way to the utilization of substances and materials which are now considered worthless. Theirs is the task of the alchemist.

Because of the rapid rate at which improvements are being made and because the application of one invention always seems to lead to several others, one wonders what is in store for the next generation. What a treat awaits those who are now students in college and what opportunities for usefulness they will have as a result of the research work which is now going on!

Most of the inventions of the future will probably not be new and basic, but will come as an evolution of those previously made. Many will operate toward the elimination of wasteful methods, making fuel, for instance, do a greater amount of work per unit. Some will enter a new field and, as an illustration, there may be cited the case of the recent development of the Flettner rotor ship which is propelled by the wind, without sails, utilizing the tendency of a rotating cylinder to create a vacuum on one side of the cylinders. It has been suggested that this principle might be applied to the development of a type of helicopter, with the idea that if the experiment were successful it would have a tendency to revolutionize the whole theory of heavier-than-air flying machines.

The habits and customs of our people as well as their social structure have been profoundly influenced by our wonderful material development. We have been changed from an essentially agricultural people to an industrial nation. When the first Federal Census was taken in 1790, probably more than 90% of the total population of about 4 000 000 was rural and less than 10% was urban, that is, dwellers in cities of 2 500 or more. Philadelphia, with a popu-

lation of 42 520, was the largest city and New York was second with 33 131 inhabitants. In 1920, only 46% of the 113 000 000 inhabitants of Continental United States was rural and 54% was living in cities or towns of 2 500 or more inhabitants. Like the advance in applied science, the change from the rural to the urban condition has been going on at an ever-increasing rate, but some of those who have analyzed the figures believe there are signs of a slowing down of this rate. The automobile, with improved highways, the telephone, and the radio, together with labor-saving farm machinery, with improved living conditions on the farm, have come to make country life more comfortable to such an extent that the migration to the cities is being gradually checked.

These changed conditions are not peculiar to our own country and Canada. Europe and the world over show the same tendency, thus indicating that the fundamental causes are not due alone to the development of a relatively new country, but to the growing substitution of machinery for hand work and the creation of new conditions of living. The change, however, is probably more pronounced in newer countries like our own.

A few examples of the recent growth of the leading cities are given in Table 1.

TABLE 1.

City.	1880.	1890.	1900.	1910.	1920.
New York, N. Y.	1 911 698	2 507 414	3 437 202	4 766 883	5 620 048
Chicago, Ill.	503 185	1 039 850	1 698 575	2 185 283	2 701 705
Philadelphia, Pa.	847 170	1 046 964	1 293 697	1 549 008	1 823 779
Boston, Mass.	362 839	448 477	560 893	670 585	748 060
Baltimore, Md.	332 313	434 439	508 957	558 485	733 826
Los Angeles, Calif.	11 183	50 395	102 479	319 198	576 673
New Orleans, La.	216 090	242 039	287 104	339 075	387 219
Seattle, Wash.	3 533	42 837	80 671	237 194	315 312

The growth of the cities, however, does not tell the whole story. In the old days, the city was little more than a layout of streets and a collection of houses. Business offices and shops were under the same roof or within walking distance of the homes of both employer and employee. The homes and shops were lighted by candles or lamps. The water supply, in most cases far from wholesome, came from wells or town pumps, and cesspools did in part the work of the modern sewer. Where paving existed at all, it was rough and generally in poor repair. Live stock roamed the streets which were cleaned only occasionally. Living, when viewed from the standpoint of to-day, was primitive. The relations between private and public health were unknown, and the police power of the body politic was confined to the simple preservation of the peace.

The modern city is a most complicated organism. Its streets and buildings are but an expression of the spirit of the city even as a photograph expresses the character of the individual, but the city itself is more than a mere expression. I have often compared the difference between the modern city and the village from which it was developed with the difference between

the modern battleship and the wooden frigate of Nelson's day. The latter was driven by the winds of heaven, the sails were set and furled, and the anchor was weighed by hand. The battleship of to-day is a bundle of nerves and in its hull is installed the most intricate machinery of many kinds. The crew knows little of the things that sailors of old had to know, but now includes specialists of many kinds, skilled in the use of steam and electrical machinery and the radio, and experts in scientific gunnery and navigation. Practically all the activities of a large town are represented on the battleship and its personnel includes opticians, dentists, physicians and surgeons, carpenters, painters, barbers, tailors, printers, even the ministry is represented. Its officers have been taught and drilled in almost every science as well as in the principles of local and international law.

The homes and offices of every modern city are equipped with electric lights and telephones and the cooking is done largely by gas, piped from a central plant. Electric cars and vehicles propelled by gasoline-driven motors take the people to and from their homes. Buildings, many stories in height and equipped with elevators, have taken the place of the old low-lying buildings. Detached dwellings are rapidly being replaced with apartment houses, each one sheltering many families. The streets are well paved and lighted and are cleaned more often and much better than they were formerly. Probably there are still as many horses in the streets as there were formerly, yet horse-drawn vehicles have been so largely supplanted by motor trucks of much greater capacity that it seems as if the horse is disappearing.

The unsanitary cesspools have been superseded by sewers which automatically carry away the wastes and the town pump has been supplanted by a wholesome water supply brought from distant sources. The newspapers are supplemented by motion pictures and by the radio, which give the people in their theaters and homes the news of the day. Religious, educational, hospital, and amusement facilities exist in great numbers, as a part of the equipment of the modern city, and are available for the poor as well as the rich to a far greater extent than ever before. The home of the laborer is fitted with comforts and conveniences that the wealthy man of the past could not command because they did not then exist. The general health of the people is better than in the more primitive days of the past. The comforts of yesterday have become the necessities of to-day and what were formerly regarded as luxuries are now demanded as comforts.

The citizen has to pay, of course, for all these modern facilities and this fact is brought to mind forcibly by some statistics which I saw recently in a copy of the "Gazeteer of the State of New York" published in Albany in 1813. The population of the City of New York, according to the Census of 1810, was given as 96 373, including 1 686 slaves. The budget of the City for 1812 gave the receipts as $1 012 460.38 and the expenditures as $953 736.04. Included in the receipts was an item of $4 969.55 for street manure. The authorized budget of the City for 1925 is nearly $400 000 000 for an estimated population of about 6 000 000. In other words, while the population has increased 62 times, the budget has increased nearly 400 times.

Seldom do we pause to think of the enormous power which has been harnessed and is available within the confines of a single large city. It is estimated that the potential power transmitted through the water, gas, and steam mains, and the electric power cables, underlying the streets of New York, is at least three times the amount now being generated by all the hydro-electric plants at Niagara Falls on both sides of the river. The capacity of the central electric generating plants in New York alone is about 3 000 000 h.p. I have been informed that the condensing water pumped for these plants is eight times the quantity of water supplied to the city by its aqueducts.

In a material sense, the modern city is the result of the work which has been done by the pure scientist and the engineer. Without them the city could not exist. To their skill and toil are due the water supply, the sewers, and the sanitary conveniences, the paved streets, the transportation facilities for local, suburban, and interurban use, the telephones, the electric lights, and all the many other facilities on which a city has come to depend for its very existence. Nor could the city be kept free from epidemics and its health be maintained at so high a standard without the excellent work of the sanitary engineer in co-operation with the chemist, the bacteriologist, and with medical research. So it has come about that typhoid and yellow fever have been largely eliminated and nearly all communicable diseases have been brought under control.

All the utilities which so largely affect our modern urban life require the services of specialists, carefully trained, each in his particular line of work. The day of the "Jack of all trades" has passed, because it is obvious that to-day, with such a tremendous amount of detail in every line of effort, one can no longer be an expert in everything and these experts are engineers by whatever name they may be called. To think of the long and patient study which has been, and is being, given to each one of these problems, to realize that the execution of the work is frequently done under the most discouraging conditions, and that progress is generally made in spite of and not with the help of the multitude, makes us feel the extent of the debt we owe to those who have labored in the laboratory and the designing room, the office, the shop, and the field, to accomplish results we accept so complacently. Few of these men are known to the public because their deeds are not of the sensational character that commands public attention and most of them are too modest to expect any public commendation for their work.

The change to urban life has affected deeply the customs, the habits, and the thoughts of the people. In the simple days during the early life of the Republic, when men lived generally distant from one another, they were largely individualists, believing with Thomas Jefferson that government was a necessary evil·and that they should get along with as little of it as possible, depending on their own efforts for success. Modern concentration of vast numbers of people in cities has led to the organization of industrial corporations with hundreds and frequently thousands of employees working under the same roof. Diversities of custom are disappearing. Organizations based

on class consciousness are formed with the idea of bettering the material condition of particular classes of workers. More government is demanded. There is a drift toward paternalism and socialism; a tendency to lean on the State or National Government for help out of all difficulties; to lose that Anglo-Saxon spirit of independence which formerly prevailed and which was the foundation on which the structure of our national life was built. The fear exists that these tendencies are going too far; that the worker is being made into a machine and that so much effort is given to developing his material side, that the moral and the spiritual sides are forgotten. While city workers have escaped the hardness of the farm life of the old days, they are paying for the ease of life in a loss of that touch of fundamental things which is so necessary for true happiness and for the full rounding out and proper balance of humanity. We have boasted that the percentage of illiteracy is much lower in our cities than in many of the rural districts, but have forgotten that education consists not only in the ability to read and write, but that its true purpose is the teaching of an understanding of the lessons which Nature teaches, to a reading of "the sermons in the rocks," and "to the dreaming of dreams of human progress and to ultimate happiness and contentment."

The price we have paid for the many advantages which life in the great cities has given us is a large one. The opportunities and advantages of urban life have not yet come to compensate humanity for the restful quiet of the open country, for the simple pleasures it affords, and for the spirit of introspection which it fosters. Their comforts and luxuries are to be enjoyed only at the expense of a certain softening of character. The joy of doing is marred by the prevalent feeling of unrest, and the ever mounting cost of living in cities is a cause of anxiety to those of moderate means. It is becoming increasingly expensive to bring foodstuffs and other supplies into large cities, principally because of the restricted terminal facilities which are, however, in part, the result of a narrow or selfish vision of the particular community.

Large cities have come to stay and will doubtless continue to exist as long as industrial conditions and human nature remain as they are and as long as gregarious man retains the desire to live and work where other men congregate. Recognizing this, the efforts of every good citizen should be directed to making cities what they should be, by each giving of his best thought to improving the quality rather than the number of inhabitants. Many are giving thought to the proposition that it would be better for the well-being of the human race if the tendency it now displays toward urban life could be checked. It is a grave question whether the future rapid growth of cities should be encouraged. It is obvious that our cities cannot continue to grow until they include the entire population. There must come a time when their growth will be checked by the inevitable process of economic laws, if thinking men do not find some way to check it before that time arrives. It does not conduce to the well-being of the people to have cities become too large. It is not economically sound to have them do so. The cost of transporting the people workward and homeward, of bringing food to them, and

of providing for them in other ways, adds a great load to an already burdensome tax not only on themselves but on the other people of the nation.

There would be less concern regarding the present and future condition of our cities if the standard of governmental growth had kept pace with that of the material growth of our cities, but it has not. Municipal government in our country still falls far short of being what it should be, but it is to be remarked that municipal government like municipal engineering is a new problem for the world. If cities had not grown so fast there would have been more time to work out this new problem. Rapid growth is not conducive to the development of the best in government. Centuries were required to develop stable systems of self-government for States and Nations with their scattered populations, while these large cities have existed but a few generations and the problems of their government are new. Then, too, the problem is complicated by the fact that in our own country, at least, the population lacks social homogeneity. The immigrant, because of the sudden change from his old standards and conditions and by reason of his inertia toward a new environment, renders the task more difficult of solution. The sociological problems which confront municipal government are staggering in their proportions. We must not lose faith if the many evils of municipal government are not corrected at once. It will require the best efforts of wise and honest men many long years to accomplish this and it is a tribute to those who have thus labored that present conditions are no worse.

I believe we are making progress in the right direction. We find discouragement when, after going upward for a time, we drop back into the trough of the wave, but I believe each trough is higher than the preceding one and the trend is ever upward. Advancement is possible only through persistent, intelligent, and co-ordinated effort on the part of all good citizens. To be both effectual and permanent, reforms in the body politic must come from within and I have faith that they will so come. From an experience in public work of more years than I care to remember, I believe the reason we are so far behind is the apathy of the so-called good citizen. Included in this category are those engineers who show no interest in the affairs of their city except to complain when things go wrong. All such are apparently content to let others run their civic affairs for them and when they do criticize their public servants they are likely, because of a lack of knowledge of the conditions, to condemn the one who has served them well instead of the one who deserves their condemnation. By so doing they work against their own best interests. Because of this apathy, because of lack of knowledge of its own affairs, and because of the habit of jumping at conclusions, the public has the deserved reputation of being a hard taskmaster for those on whom it has conferred the responsibility of directing public business. Many are misled by the mouthings of demagogues and do not take the time to reason out the particular thing which may be under discussion. Frequently, investigations are made of the conduct of public departments, sometimes for partisan reasons and political effect. To those familiar with such matters it is known that such investigations are usually far from being thorough, that they bring out

only one side of the case, and that their conclusions, usually based on prejudice, are often fundamentally in error. The inevitable result in such cases is that the faithful and honest official, innocent of wrong-doing, is censured and discouraged. His department is disorganized because nothing is more demoralizing than a sense of injustice done. Every honest man chafes under such conditions and most men are honest. But, worst of all, the body politic suffers in such a case because whatever faith it may have had, has been destroyed. If public opinion was sufficiently alert to demand that all such investigations be rigidly thorough and impartial we would find ourselves a long step in advance. That is the way in which I believe engineers would conduct such investigations. They would bring reason to bear on them and would assemble and analyze the facts before reaching a conclusion.

I have referred to the part the engineer has played in the upbuilding of the modern city and of the debt the public owes him, but, if he has done all these wonderful things, he has assumed a responsibility for them and owes a debt in turn. We cannot bring something into being without assuming responsibility for its proper development and for the use that is made of it. After all, works of the engineer's creation are but the means to an end, not the end itself. They contribute to the well-being of humanity and man is more than a physical being. Surely, the engineer has a duty to perform in addition to the development and care of material things, wonderful as they are. I do not like to think that the engineer can see only the steel and masonry structure he designs and builds. I hope he has a vision of what it is built for as the architect of the Middle Ages had of the great cathedral he erected, and that the spiritual, moral, and æsthetic sides of life have a great value to him.

We are justly proud of what the engineer has done and perhaps we have boasted about and given too much thought to his achievements, leaving to others the concern over the grave social and political problems which confront us with those great concentrations of population in our cities. How may he help in the working out of these problems? First of all, as a good citizen, he must do his share in all branches of community work. If he does less he fails in his duty. The engineer is no longer the pioneer of the early days when his work called him to more or less remote sections to build canals or to the frontier of civilization to build railroads. Lacking a permanent abode, there may have been an excuse for neglecting his civic duties in those days, but not in these. There is no frontier with us now and as the engineer has, generally speaking, acquired a settled residence, he should take part in the affairs of the community in which he resides. By this I do not mean that he should enter politics as that expression is commonly understood. There are many ways to help besides that and many are finding the way, serving, as some are now doing, on school boards and other public and quasi-public boards to good effect and aiding in the solving of the multitude of vexing questions which arise. There is much civic work of a special nature that the broad-minded engineer is particularly qualified to assist in doing, such as the framing of building codes, zoning regulations, and of legislation affecting the regulation of public utilities.

It is not enough that he should understand his own problems. He should be able to explain them to laymen in language that is understood by them. Too often, I am afraid, the engineer fails to win approval of the authorities or of the public to a sound and meritorious proposition because he lacks the ability to translate his own sound thoughts into language that others who are not engineers will understand.

Do not let me be understood as claiming that engineers are superhuman beings, that they are made of better clay than others, or that they alone are responsible for all the good that has been accomplished. Like the rest of mankind they are not infallible. Others have been and are doing their duty to the world in their respective lines of endeavor; but it must be remembered that the engineer is trained to deal with fundamental things. He is accustomed to delve for the truth and to reject that which is unsound. Unless there is a foundation of truth in a proposition he will instinctively oppose it. Habit of thought inclines him to reason a thing out from cause to effect. Popular clamor and newspaper headlines do not sway his judgment. He is trained to look broadly and not parochially on all propositions involving the application of natural laws. Artificial political barriers do not appeal to him as they do to those who consider all things superficially, because he recognizes that the laws of Nature operate in the same way on each side of every State, National, or other political boundary.

For the engineer to bring these qualities to the public service he must in his civic associations be something more than the mere technician. I believe the engineer recognizes more than other citizens, and by force of example, by precept, and by teaching, he must show all men, that the days of waste are passing and the era of conservation has begun. With the waste must go all petty and partisan politics with so-called "log-rolling" and in its stead must be substituted constructive statesmanship to go hand in hand with the principles of conservation. If we are to live up to the ideals of our institutions our legislators must be leaders rather than followers. It has often been said that America is law-ridden and that its people have lost respect for its laws. A foreign critic said some time ago that we have more laws than all the other countries of the world combined and that we are the most lawless of all people. Is there not force in this criticism? Why should not engineers join with other good citizens to correct some of the evils of the body politic we all recognize and which are due largely to the apathy of our people? The engineer can be a good citizen without losing his value as a technical man. He yields to none in respect and affection for the institutions of his country. His patriotism was shown by his work in the World War and in many other ways. Through active participation in civic affairs his vision would be broadened and the criticism that has so often been made as to the narrow outlook of the engineer on the affairs of the world would soon fall of its own weight.

As we read day after day the sensational matter that is called news and listen to the harangues of the popular orator and self-constituted regulators of the world we wonder if they represent the standard of the character and of the intelligence of our people. Should we so believe we could easily become

discouraged as to the future of our institutions and even of civilization itself. Faith and courage will return if we realize that these are but manifestations of a small and vicious minority who bask in the sunshine of publicity. The great mass of all thinking men do their work quietly and without ostentation wherever right and duty call. In industrial plants, railroads, public works, schools and colleges, churches, hospitals, and on the farms quiet thinking men are doing the real constructive work of humanity and of civilization. With an instinctive reliance on and faith in the integrity of human nature, and believing in the permanency of those institutions of man which make for the advancement of knowledge and good of the world, they make their daily sacrifice to duty. They are representative of that divine force of progress which is irresistible in action because it is based on truth, on reason, and on character. Cannot some way be found to employ this force in the interest of civic betterment? All these workers are of the company of the engineer. With him they must advance to wider fields of greater effort, of greater promise, and of even greater service.

At the Centenary Celebration of the founding of The Franklin Institute, Dr. Arthur D. Little delivered an address on "The Fifth Estate" which he described as being "composed of those having the simplicity to wonder, the ability to question, the power to generalize, the capacity to apply. It is, in short, the company of thinkers, workers, expounders and practitioners upon which the world is absolutely dependent for the preservation and advancement of that organized knowledge which we call Science." While those who live in this estate may have the capacity to apply, we see it demonstrated on every hand that they have failed to make use of their ability. My plea goes out to those who, having the capacity to do, have failed to perform. To those of you who have not read Dr. Little's masterly address, fascinating in the beautiful simplicity of its language, I would commend it, and I know if you read it once, you will read it again. Quoting from it further:

"We see, in the ranks of science, knowledge without power, and, in politics, power without knowledge. An electorate, which regards itself as free, listens to the broadcast noise of manufactured demonstrations and is blind to the obvious mechanics of synthetic bedlam. The result is too often government by gullibility, propaganda, catchwords, and slogans, instead of government by law based on facts, principles, intelligence and good-will.

Thus, contrasting the difference between ability and accomplishment, engineers and men of scientific attainment must needs take to themselves the lesson that definite results obtained are a truer measure of the man than mere theoretical ability. The world will ever measure by performance. To it ability undemonstrated is synonymous with non-existence.

Thanks to the wisdom of our forefathers, the foundations on which our country's institutions rest are broad and deep. Incorporated in the design were high ideals and the superstructure was raised in fidelity and in faith. The task of its maintenance is that of each succeeding generation. As in the case of all structures built by man, repairs, renewals, and additions are necessary, but do not call for destruction of the original fabric. The skeleton of the structure must be maintained intact. The larger duty of the technical

man calls him to this task. To the task he brings special qualifications. As a practical idealist his patriotism and civic spirit should be manifested not by a waving of flags and by boasts of superiority to others, but by searching out the defects in our present make-up and then by aiding others to raise the standards of civic affairs in the same deliberate and constructive way in which he designs and builds his physical engineering structures for the use and benefit of mankind.

MUNICIPAL WATER SUPPLY PROBLEMS OF ATLANTA, GEORGIA*

By Paul H. Norcross,† M. Am. Soc. C. E.

Synopsis

The water supply of the City of Atlanta, Ga., is of interest from several angles, as it is the largest development of a surface water supply in the Southeastern States, furnishes a water unusually free of dissolved solids and, therefore, peculiarly well suited for industrial uses, and contains, in addition, some unique features of design and operation.

The supply is derived from the Chattahoochee River, a Cherokee Indian name meaning the "Chatter", or "Hooting of the Owl". Its source is on the eastern slopes of the Blue Ridge Mountains, at elevations ranging from 3 000 to 4 000 ft., and it flows in a southwesterly direction, through a basin singularly free of pollution.

The supply works consist of a 48-in. intake conduit, a steam-operated river pumping station, having a capacity of 106 000 000 gal. per day, pumping against a maximum head of 275 ft., including friction, through three parallel conduits, 30, 36, and 48 in. in diameter, respectively, each approximately 18 500 ft. long, into two raw water or settling basins, having a combined capacity of 393 000 000 gal.

From the raw water basins, the water flows by gravity through a battery of five coagulating basins, having a combined capacity of 13 000 000 gal., and then to two gravity filtration plants, operated in parallel. One plant, the original installation, is of the pressure type, containing 48 units, 12 vertical and 36 horizontal, having a total rated capacity of 21 000 000 gal. daily, and the other, just completed, a plant of equal rating of the reinforced concrete gravity type, having 7 units, making a total combined rated capacity of 42 000 000 gal. daily.

From the filtration plants, the water passes through a 60-in. conduit, to a new reinforced concrete covered reservoir, of 10 000 000 gal. capacity, which is cross-connected to a steam-operated high-service pumping station, having a rated capacity of 105 000 000 gal. per day, delivering water direct to the distribution system against a maximum head of 100 to 120 lb.

Some of the features of design and operation include both inverted and overhead siphons of the raw water conduit, either in tunnel or aqueduct; flexible operation of the raw water conduits, raw water basins, etc.; a new, open-gallery type filtration plant and appurtenances, and a new reservoir and

* Presented at the meeting of the Sanitary Engineering Division, Atlanta, Ga., April 10, 1924.

† Cons. Engr., Atlanta, Ga. Mr. Norcross was drowned at the time of the sinking of the *Norman* in the Mississippi River on May 8, 1925.

turbo-centrifugal pumping units that have developed unusually high efficiencies for loads varying from 50 to 110 per cent.

The successful and economical operation in parallel of two widely different types of filter plants, is worthy of comment.

The problem of keeping pace with increasing demands, due to the rapidly growing population and industrial, commercial, and municipal consumers, while maintaining service, is being accomplished without interruption thereof.

INTRODUCTION

The purpose of this paper is to describe briefly the history and development of Atlanta's municipally owned water utility which, because of its importance and certain unusual features of design and operation, seems worthy of attention, and also to stimulate renewed interest in the broad field of water treatment. This subject is of increasing importance, particularly in the southeastern section of the United States which, due to its equable climate, wealth of raw materials, wonderful agricultural possibilities, and growing economic advantages, is gradually assuming supremacy in many fields, including the textile industry, and where numerous other classes of manufacturers and commercial concerns are prospecting for locations.

The extent of domestic and industrial use of public water supplies in the United States was the subject of a recent investigation by the United States Department of the Interior, and the result of this study is set forth in a paper entitled "Utility of Public Water Supplies in the United States",* which publication is a most valuable addition to water supply data. This paper serves to emphasize the value of water free of minerals especially in industrial lines, including boiler plants, and "the manufacture of soft drinks, bakery products, and other foods." As Atlanta is the "birthplace" of probably the most widely known "soft drink" manufactured to-day, the significance of this statement is obvious.

It is, of course, needless to emphasize the importance of a high sanitary quality for public water supplies, or to review the decrease in "typhoid toll", due to advance in the science of water treatment. It is worthy of note, however, that the growing and more exacting demands of the consumer of to-day is for water not only free of suspended matter and polluting organisms, but also one more acceptable from the standpoint of mineral content.

The distribution of "hard" and "alkali" waters of various degrees of hardness throughout the country is general, occurring in twenty-seven States, comprising probably 75 to 80% of the total area of the United States. The localities providing so-called "soft water" are confined to nineteen States located in three groups, situated in the Northwest, Northeast, and Southeast, respectively, with the last predominating in area. Georgia occupies the geographical center of this latter area. That part of those States in the Southeast above the coastal plain obtain their supplies principally from the eastern and southeastern slopes of the Blue Ridge and Appalachian Mountain Ranges,

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* *Water Supply Paper No. 496*, U. S. Geological Survey.

and Atlanta's supply ranks foremost in size and character of development in this group.

It is not intended to review in detail or to describe, except briefly, those parts of the utility which are commonplace, but it is the desire of the writer to present a comprehensive view of the problem and to describe such parts of the development as seem interesting and unusual.

HISTORICAL

The real history of Atlanta's water utility originated in 1872, when the first Water Commission of the city contracted for the construction of a water supply on the head-waters of a stream having its source within the city limits, flowing in a southeasterly direction, and being a tributary of the Altamaha Drainage Basin, with its outlet in the Atlantic Ocean.

This supply consisted of an impounding reservoir on the head-waters of South River, about 3 miles south of the center of the city, of 250 000 000 gal. capacity, covering an area of 51 acres, with a drainage area of approximately 5 sq. miles.   The water from this supply was originally untreated, but, about 1886-87, Hyatt pressure type filters were installed (probably one of the first pressure type installations in the United States), from which the water was collected in a clear-water basin having a capacity of 800 000 gal. and then pumped direct to the distribution system by steam-driven Holly quadruplex type pumps, one of 2 000 000 and the other of 4 000 000 gal. daily capacity.

This supply, the first water from which was pumped into the mains on September 8, 1875, was supplemented in 1884 by an Artesian well, located in the center of the city, which was drilled through about 2 000 ft. of granite, or gneiss—the underlying bed-rocks of Atlanta. This well produced a limited supply of water from four veins, 102, 225, 450, and 1 160 ft. below the surface, respectively.

Records of water used daily and annually prior to 1888 are very meager, but the total quantity pumped from South River during that year was 629 000 000 gal., supplemented by 10 500 000 gal. from the Artesian well supply, which latter quantity represented the total available from this source. It is interesting to note that all domestic consumers were metered in 1888, and that a uniform charge of 17 cents per 1 000 gal. was made.

As a part of the drainage area of South River was within the city limits, and as unusual growth of the city began between 1880 and 1890, it was soon obvious that South River water, although originally of unusually good quality, was becoming grossly polluted. In addition, the supply available from this stream, with its limited drainage area, was not adequate to meet the existing and increasing demands made upon it. It, therefore, became necessary to obtain another and more ample supply.

In 1889, the city officials engaged the late Rudolph Hering, M. Am. Soc. C. E., to make surveys, investigations, and report on a new supply. Mr. Hering's report submitted under date of October 15, 1889, recommended the development of a supply from the Chattahoochee River. This stream has a drainage area of about 1 500 sq. miles with its head-waters in the Blue Ridge Mountains of North Georgia. It is a tributary of the Apalachicola Drainage

Basin and flows in a southwesterly direction into the Gulf of Mexico, which course brings it, at its nearest point, within about $6\frac{1}{2}$ miles northwest of the center of the city. This stream, although carrying large percentages of turbidity during certain periods and nearly always highly colored with clay and silt, was of splendid quality, with only nominal pollution and with a chemical analysis indicating a water almost free of mineral content.

The analysis of Chattahoochee River water, as made by the U. S. Geological Survey, is shown in Table 1.

TABLE 1.—ANALYSIS OF CHATTAHOOCHEE RIVER WATER, IN PARTS PER MILLION.

Total dissolved solids.	Loss on Ignition.	Silica, SiO_2.	Iron, Fe.	Calcium, Ca.	Magnesium, Mg.	Sodium and potassium, Na = K.	Bi-carbonate radical, HCO_3.	Sulphate radical, SO_4.	Chloride radical, C l.	Nitrate radical, NO_3.	Total hardness as Ca Co_3.
32	1.5	10	0.08	2.4	1.1	Na, 2.3 / K, 0.6	6.6	7.8	1.8	0.33	11

The topography of Atlanta is peculiar and interesting. It consists of a series of ridges averaging 950 to 1 000 ft. above sea level, that divide the drainage of streams flowing either into the Atlantic Ocean or the Gulf of Mexico. This is an advantage from a sanitary standpoint, but the deep valleys and high ridges naturally affect the pumping problem and add to the cost.

Mr. Hering recommended the construction of a pumping station on Chattahoochee River, with an intake at the point at which it is at present located, just above the confluence of Peachtree Creek, as this latter stream drained a large part of the area within the city limits, and the construction of raw water storage reservoirs, treatment plant, and a high-service pumping station about half way between the river and the center of the city. The point on the river selected for development was about 300 ft. lower in elevation than the highest ridges in the city, thus making necessary two pumping stations to provide adequate pressures. Table 2 gives the elevation of points in the city and on the river.

TABLE 2.

	Feet above sea level.
Low water, Chattahoochee River, at mouth of Peachtree Creek...	758
Raw water reservoir site...	985
High levels in city...	1 050-1 065
Ordinary level of domestic pressure................................	1 100-1 150
Fire-pressure level..	1 200-1 250

The wisdom of selecting the Chattahoochee River as the source of the ultimate and final supply for Atlanta has been proved. The character of the region at its source, consisting of unpopulated mountain ranges between 3 000 and 4 000 ft. above sea level, with unusually high annual rainfall, and the

fact that it flows through a sparsely settled area and drains practically no towns or cities of size, combined with the excellent chemical quality of its water, are factors in its favor.

Fig. 1 shows the drainage area of the Chattahoochee River at the intake. It will be noted that no large towns are in its basin. The diagram also shows the location of the South River development—the original public water supply of Atlanta—and the present outlines of the city limits.

ORIGINAL PLANT

The City began the development of Chattahoochee River in 1891 and placed the new supply in service in September, 1893. The original development consisted of a gravity-operated, 48-in. cast-iron pipe intake about 700 ft. long, discharging into an open intake well, and a brick pumping station surrounded by an earth dike to prevent flood overflows. Steam-driven Holly horizontal compound pumps, operated against a suction lift of about 16 ft., delivered the water through a 30-in. cast-iron main, about 18 500 ft. long, to an unlined raw water storage reservoir of 177 000 000 gal. capacity constructed in cut and fill and by damming several ravines.

The water was then passed by gravity through concrete coagulating basins to a battery of horizontal pressure type filters, operated as closed gravity filters with an average operating head of 30 ft., and thence to an uncovered clear water well. The total loss of head between high-water level in the raw water reservoirs and the high-water line in the clear water well was 40 ft. From there, the water was conducted to the high-pressure pumping station known as the Hemphill Avenue Station. The reservoirs, coagulating basins, filter plant, and pumping station were located on the main line of the Southern Railway about three miles northwest of the center of the city. This station was similar in type, design, and equipment, to the South River Station.

It is interesting to note that the old Hyatt filters, twelve in number, which were of the vertical pressure type, were removed from the South River Station and installed, and operated in parallel, with the horizontal filters at the new plant.

PLANT IN 1920

Between 1893 and 1920, this plant was expanded along the lines of the original development and, in 1920, the pumping equipment at the Chattahoochee River Station consisted of two 10 000 000-gal. Holly horizontal compound pumps, one 18 000 000-gal. Holly horizontal compound pump, one 10 000 000-gal. Worthington motor-driven centrifugal pump, and one 28 000 000-gal. DeLaval turbo-centrifugal pump, the last having been installed in 1918-19, to meet emergency war demands due to the construction of Camp Gordon, near Atlanta.

The raw water conduits consisted of the original 30-in. and an additional 36-in. cast-iron main, delivering water against a head of 260 to 290 ft., including friction, to two raw water reservoirs with capacities of 177 000 000 gal. and 216 000 000 gal., respectively.

The treatment plant consisted of five coagulating basins having a combined volumetric capacity of about 13 000 000 gal., and the filter plant con-

tained thirty-six horizontal type pressure units of 500 000 gal. daily capacity each, and twelve Hyatt vertical type pressure units with rated daily capacities of 250 000 gal. each, making a total combined rated capacity of 21 000 000 gal. daily, discharging into two circular, open-type, clear water wells having a combined available capacity of 2 000 000 gal.

The Hemphill Avenue, or High-Service, Station, contained two 10 000 000, and one 15 000 000-gal. horizontal compound pumps, one 20 000 000-gal. vertical triple expansion pump, and one 30 000 000-gal. DeLaval turbo-centrifugal pump installed in 1920.

A brief résumé of the demands of service and the capacities of several of the elements composing the plant in 1920 should aid in a clear understanding of the major problems that confronted the city at that time. These are shown in Table 3.

TABLE 3.

	Gallons daily.
Average rate of consumption (1920)...........	24 000 000
Maximum rate of consumption (1920).............................	40 000 000
Maximum capacity of supply mains from river (1920)...............	33 000 000
Rated filter capacity (1920)...	21 000 000
Clear-water storage (1920)...	2 000 000

In addition to the fact that parts of the plant were wofully inadequate in size and arrangement to meet existing demands, much of the equipment had reached the end of its useful life, or was inefficient and obsolete.

INVESTIGATIONS AND STUDIES

In 1920, the City of Atlanta authorized complete surveys and investigations to determine the necessary improvements and extensions to the water supply of the city. This investigation included study of the river intake, the pumping station at Chattahoochee River, supply mains between the river and raw water reservoirs, coagulating basins, filter plant, and high-service pumping station, including all connections and appurtenances. These studies were started in the latter part of 1920 and extended through 1921 and, as a result, recommendations were made, which briefly were as follows:

(a).—Construction of an impounding dam, new intake, and pumping station to supplement the existing intake and station.

(b).—Remodeling of the Chattahoochee River Station, including additions and changes in the boiler plant, remodeling of steam headers, new chimney, installation of new and efficient pumps, and the re-arrangement of suction, discharge lines, and headers.

(c).—New raw water supply main from the River Station to the raw water reservoirs, paralleling the existing 30 and 36-in. mains.

(d).—Installation of gate-house and cross-connection, controlling and making more flexible the operation of the raw water reservoirs.

(e).—Construction of a mixing chamber and a new chemical house for the proper treatment of water prior to coagulation and filtration.
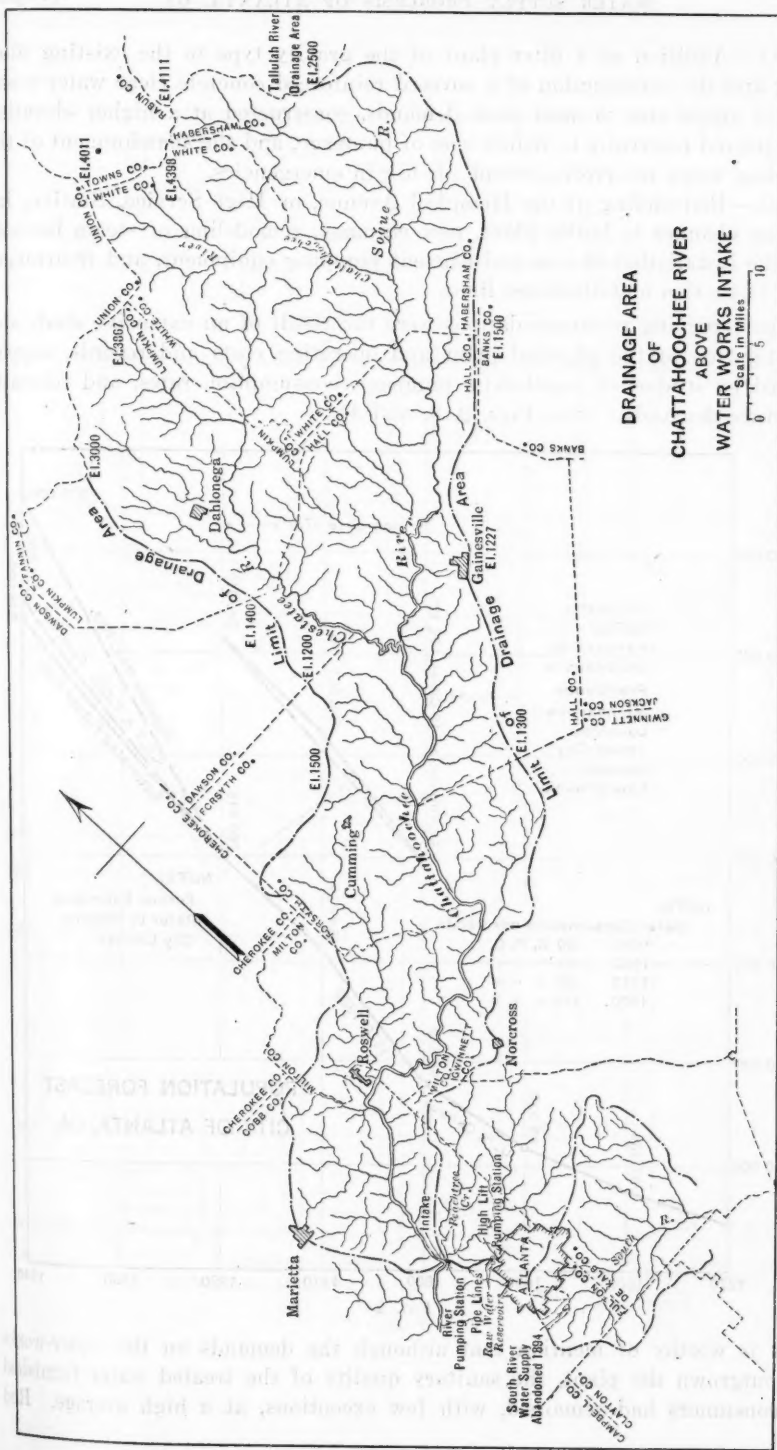
DRAINAGE AREA
OF
CHATTAHOOCHEE RIVER
ABOVE
WATER WORKS INTAKE

Scale in Miles

FIG. 1.

(*f*).—Addition of a filter plant of the gravity type to the existing filter plant and the construction of a covered reinforced concrete clear water reservoir of ample size to meet peak demands, constructed at a higher elevation than the old reservoirs to reduce cost of pumpage, and the abandonment of the old clear water reservoirs, except for use in emergencies.

(*g*).—Remodeling of the Hemphill Avenue, or High-Service, Station, including changes in boiler plant, new chimney, remodeling of steam headers, and the installation of new and efficient pumping equipment, and re-arrangement of suction and discharge lines.

The foregoing recommendations were the result of an extensive study and investigation of the physical plant and operating costs and records, supplemented by studies of population, pumpage, consumption rates, and forecasts of future demands. (See Figs. 2, 3, and 4).

POPULATION FORECAST
CITY OF ATLANTA, GA.

Rochester
St.Paul
Kansas City
Indianapolis
Providence
Minneapolis
Louisville
Jersey City
Newark
New Orleans

NOTE:
Water Consumption per Capita
1890   40 G. P. C.
1900   68 "   "   "
1910   80 "   "   "
1920   120 "   "   "

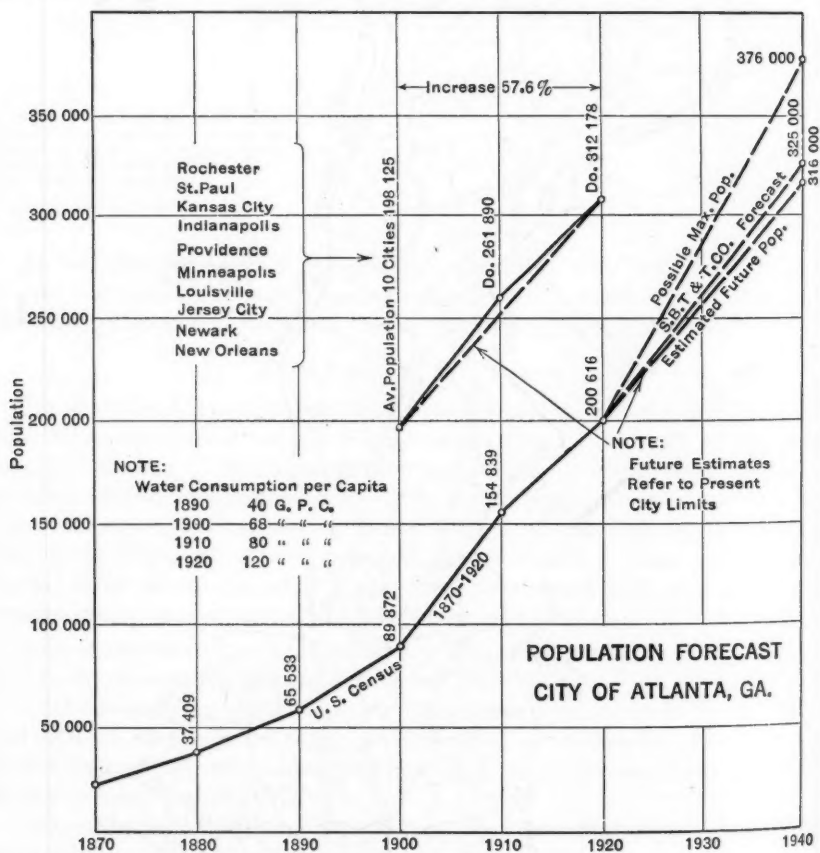NOTE:
Future Estimates
Refer to Present
City Limits

Fig. 2.

It is worthy of mention that although the demands on the water-works had outgrown the plant, the sanitary quality of the treated water furnished the consumers had remained, with few exceptions, at a high average. Red
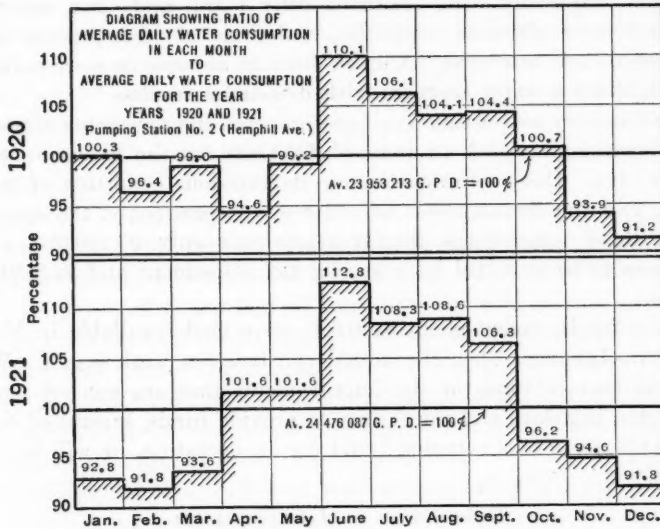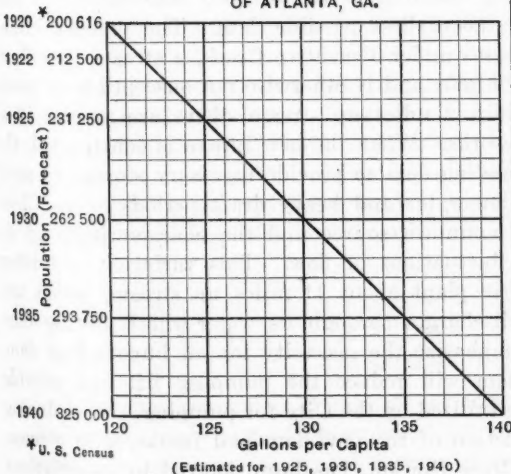
DIAGRAM SHOWING RATIO OF
AVERAGE DAILY WATER CONSUMPTION
IN EACH MONTH
TO
AVERAGE DAILY WATER CONSUMPTION
FOR THE YEAR
YEARS 1920 AND 1921
Pumping Station No. 2 (Hemphill Ave.)

1920

Percentage

110.1
106.1
104.1 104.4
100.3 99.0 99.2 100.7
96.4 94.6 93.9
91.2

Av. 23 953 213 G. P. D. = 100%

1921

112.8
108.3 108.6
106.3
101.6 101.6
92.8 91.8 93.6 96.2 94.6
91.8

Av. 24 476 087 G. P. D. = 100%

Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.

FIG. 3.

ESTIMATE OF
FUTURE PUMPING REQUIREMENTS
OF ATLANTA, GA.

Population (Forecast)

	HEMPHILL STATION			RIVER STATION
	Av.	Av. Max.	Av. Min.	
	M.G.D.	M.G.D.	M.G.D.	M.G.D.
1920 *200 616	23.95	33.53	14.37	
1922 212 500	25.92	36.28	15.55	28.8
1925 231 250	28.90	40.46	17.34	32.11
1930 262 500	34.13	47.78	20.48	37.92
1935 293 750	39.65	55.51	23.79	44.05
1940 325 000	45.50	63.70	27.30	50.55

120   125   130   135   140
Gallons per Capita
(Estimated for 1925, 1930, 1935, 1940)

*U. S. Census

NOTE:
Estimates under "River Station" include
Allowance for Leakage, Evaporation, Etc.
NOTE:— "Av. Max." = 140%
         "Av. Min." = 60%

FIG. 4.

or colored water, however, was becoming more frequent, and it was obvious that the intake facilities, raw water pumping plant, raw water supply mains, pre-filtration and coagulating facilities, filter plant, and clear water storage had reached their ultimate capacities, and that the high-pressure pumping station needed early attention. A break-down at any one or more points would have precipitated a water shortage with disastrous results.

The preliminary cost estimates of the proposed improvements approximated $4 600 000, which included an item of $150 000 for the extension of mains within the city. Due to the customary impecunious condition of municipal treasuries, and the crying needs of other city departments, the amount provided for these water-works improvements was only $3 100 000, of which $250 000 was to be provided by a special tax assessment and $2 850 000 by a bond issue.

All these funds, except about $100 000, were made available in November, 1921, and in January, 1922, the actual construction work began. With the exception of certain items of the improvements that are not yet under construction, due to delays and the failure to provide funds, several of the major improvements are either completed and are in operation, or will be within a short time.

DESCRIPTION OF IMPROVEMENTS

New Intake and Impounding Dam.—The present intake which consists of one line of 48-in. cast-iron pipe terminating in a brick pier, with iron grating inlets, located in the bed of the river, is not submerged completely at all periods, as low-water conditions often partly uncover it. The normal water level of the stream has been raised by constructing a rip-rap and a pile dam, but due to increasing demands, it is extremely important that new intake works be provided at the earliest possible date. The present intake line is poorly located and crosses under Peachtree Creek, a stream that flows into the river just below the intake, and is otherwise not susceptible of much improvement. However, within 2 miles up stream, there are several ideal locations for the new intake works. When the new intake is constructed, the City will also build an impounding dam to provide necessary storage to meet future requirements during low-water and heavy draft periods. The Chattahoochee River, due to its mountain source and the characteristics of its basin and run-off, has wide fluctuations of flow. This variation is further complicated by a hydro-electric plant about 11 miles up stream, which impounds water at night and discharges through its water-wheels during daylight hours. These factors emphasize the necessity for an impounding dam. The head created by this dam will reduce the pumping lift and provide hydraulic power which will be utilized by the City for pumping, thus reducing operating costs. The construction of the new dam and intake, with connections to the Chattahoochee River Station, has been delayed by negotiations between the City and riparian owners and the usual lethargy of municipalities in providing the necessary funds.

It is proper to state here that the selection of the original intake and pumping station sites was probably influenced by considerations of economy and

expediency. The fact remains, however, that the present arrangement and design preclude additional expansion along good engineering lines. The location of the pumping station about 25 ft. below the flood-plain of the river, thus requiring the maintenance of dikes and emergency attention to prevent flooding through the intake well at high-water periods, supplemented by the fact that the sole dependency for supply is on one single inlet separated from the pumping station by an intervening stream, is a condition requiring prompt correction.

Remodeling Chattahoochee Station and Appurtenances.—To provide capacities, economies, and flexibility, and, at the same time, maintain service, it was found necessary to make radical alterations and extensions and additions to the Chattahoochee Station. These improvements consist of a new radial brick chimney, changes and additions to the boiler plant, re-arrangement of steam headers, abandonment and removal of the two 10 000 000-gal. pumps and the one 10 000 000-gal. motor-driven unit, installation of efficient pumping units, remodeling of suction and discharge lines and headers, and the reconstruction of the interior of the pumping station.

Practically all this work has been accomplished without interruption of service, and the present equipment of the Chattahoochee Station consists of the following:

One boiler plant, having a rated capacity of 1 850 h.p.
One 18 000 000-gal. compound condensing Holly unit (installed in 1904).
One 28 000 000-gal. DeLaval turbo-centrifugal unit (installed in 1919).
Two 30 000 000-gal. DeLaval turbo-centrifugal units (installed in 1918-19).

All the pumps lift water from the open suction well and discharge either into a new 48-in. header, or directly into the raw water supply mains.

The two 30 000 000-gal. turbo-centrifugal units were purchased in competition with 15 000 000-gal. turbo-centrifugals and reciprocating units. The load on Chattahoochee Station is constant and susceptible of little variation, consequently, the use of smaller units represented no economy. The guaranties made by the successful bidder were based on the use of steam at 175 to 190 lb. pressure, with 100° superheat, and are as follows:

110% load......163 400 000 ft-lb. per 1 000 lb. of steam
100% load......166 000 000 "   "   "   "   "   "
 75% load......151 000 000 "   "   "   "   "   "
 50% load......126 900 000 "   "   "   "   "   "

They were completed in the fall of 1923. Duty tests were made on them during the latter part of December, 1923, and in January, 1924; in all these tests they have met or exceeded their guaranties and are notable examples of recent development in this type of equipment.

The re-arrangement of the suction and discharge lines provides ample facilities for flexible operation and increased demands. These lines are arranged with the idea of preventing interruptions of service from accidents and other emergencies. In addition, the station is provided with indicating

and recording instruments to facilitate efficient operation, and the keeping of proper records.

The present intake facilities have not been increased, principally because the arrangement and physical location are poorly suited for further development. The capacity of the 48-in. cast-iron intake is limited by its size and the head of water over its inlet, which head, as previously stated, varies not only during the wet and dry seasons, but fluctuates daily and hourly due to the existence of the hydro-electric plant of the Georgia Railway and Power Company at Morgan Falls, several miles up stream, which stores the water at night and discharges during the day.

The problem of increasing the water level over the intake is complicated by the existence of a shifting sand-bar, or island, in mid-stream, known as DeFoor's Island, just opposite the water-works plant. In years past, the elevation of the water over the inlet of the intake has been increased by the construction of a rip-rap dam between the east bank of the stream and the Island, and a pile dam with rock-fill, between the north point of the Island and the west bank of the stream. During 1924, additional rip-rap will be added to the existing dams to increase the water level, but this will provide only temporary relief, as it will be necessary to supplement this intake by the construction of another one which will be located up stream at a more suitable site.

New 48-In. Cast-Iron Raw Water Main.—The necessity for new raw water supply conduits was emphasized by the fact that the peak demand which was increasing, exceeded the combined daily capacities of the existing 30 and 36-in. cast-iron mains. Detail surveys of existing and available routes and study of the future demands showed the advisability of installing an additional raw water main not less than 48-in. in diameter, along the same route as the existing mains from the Chattahoochee Station to the raw-water reservoirs.

After receiving competitive bids on steel, reinforced concrete, and cast-iron conduits, the last was selected. The conduit was constructed in sections and placed in operation as each section was completed. It is now finished and in operation. Construction of this line was somewhat complicated and delayed by the necessity of crossing the tracks of three railroads and two interurban electric railways. Inverted siphons were used under two of the railroads, one siphon crossing eleven tracks and the other nine. The problem of maintaining water-tight conduits was perplexing and constituted a perpetual hazard with an economic loss due to constant leakage and occasional breaks.

The delay in the completion of these railroad crossings was due to a long series of conferences with the railroad officials as to the merits and costs of underground conduits in open cut or tunnels, and overhead crossings, and the existence of previous contracts between the City and the Railroad Company, which required the Railroad Company to pay for maintenance. Eventually, new and equitable contracts were executed, and the remaining portion of the new 48-in. main, between the Chattahoochee River and the raw water reservoirs, is now complete. Fig. 5 is a view of the pipe gantry used in laying the 48-in. cast-iron main.

Some of the more interesting details of this main include the construction of a differential surge tank at the high point on the line, with individual connections to the 30, 36, and 48-in. mains; and the elimination of one of the inverted siphons of both the existing and new mains by the construction of an overhead aqueduct, consisting of a steel span with steel girder approach on one end and a reinforced concrete span on the other end, carrying two lines of 48-in. cast-iron pipe, over the tracks of the Seaboard Air Line Railroad. Fig. 6 shows the differential surge tank and a view of the raw water mains.

The new main is also cross-connected with the 30 and 36-in. mains at four intermediate points with numerous fittings and gate-valves, which permit economical repairs with a minimum loss of time and also provide flexibility of operation.

Gate-House and Cross-Connections, Raw Water Reservoirs.—Previous to 1921, the physical connection between the raw water conduits and the two raw water reservoirs was very limited. To provide flexible operation of the two basins and also permit the lowering of the level of one or both without undue delay, new cross-connections with a centrally controlled gate-house were planned. This was all the more necessary in view of the increased capacities that were being provided by the installation of the new 48-in. raw water main.

As designed and constructed, the gate-house located between the two reservoirs, receives the water from the three raw water conduits through two 48-in. inlet pipes discharging into a common well of reinforced concrete construction. This well is cross-connected by a cast-iron pipe conduit 60 in. in diameter to the raw water reservoirs, the invert of which is 7 ft. below the normal high-water surface of the basins.

The raw-water inlets and the outlets to the two reservoirs are controlled by two 48-in. and two 60-in. sluice-gates, electrically operated, with hand-wheel connections for emergency, all of which is housed in a substantial brick building.

Mixing Chamber and New Chemical House.—The pre-filtration processes of water treatment have grown in importance and magnitude in recent years. More extensive and detail operating records, supplemented by technical data obtained from numerous tests and investigations stimulated by various agencies, have emphasized the necessity for more scientific and comprehensive methods of preparing the water for filtration.

The Chattahoochee River carries variable and, sometimes, large quantities of suspended matter so finely divided that it will not settle even after a long period. It is necessary, therefore, to treat the water with a coagulant, which facilitates the removal of this finely divided material in coagulating basins. The process generally consists of injecting a solution of aluminum sulfate which due to the normal alkalinity of the water produces the usual flocculent precipitate and deposition of the suspended matter. Good coagulation cannot always be obtained by the use of this chemical alone, as chemical and physical changes in the water, supplemented by the large range of turbidity content

due to heavy rainfalls, etc., prevent the adequate formation of the flocculent precipitate and, consequently, the delivery of unclarified water to the filters. In many instances these changes take place rapidly. The effect of these changes are of more importance particularly in water similar to that of the Chattahoochee River, containing a small quantity of dissolved solids.

These varying factors affecting the Chattahoochee River water, including reduction in alkalinity content and intensity of acidity—expressed as the hydrogen-ion concentration—supplemented by high turbidity, have often caused the delivery of discolored and unsatisfactory water to the distribution system, as the failure to clarify the water properly in the coagulating basins and the delivery of partly treated water with a large quantity of suspended matter to the filters, interferes with their normal operation.

The reaction between aluminum sulfate and the carbonate or bi-carbonate of natural waters often sets free a quantity of carbon dioxide, thus making the intensity of acidity great enough to cause corrosion in iron pipes and consequently red water. This is unfit for use by both the domestic and industrial consumer and will eventually damage the pipes.

The facilities for chemical treatment in 1920 were housed in a small brick building, at one end of the coagulating basins. In addition to the chemical and bacteriological laboratory, this building housed two wooden tanks for chemical solutions and provided storage space for alum, lime, etc. Normally, the alum solution was delivered by gravity through two parallel lines of lead pipe about 500 ft. long, to the 60-in. raw water conduit conducting the water from the raw water to the coagulating basins. Facilities for providing artificial alkalinity consisted of emergency preparations of solutions of soda ash, or lime, in one of the chemical tanks. This "hit-or-miss" method, without properly designed structures and equipment, has been the source of much trouble in the treatment of water at Atlanta. In 1921-22, the City installed a dry feed machine for applying lime, at a point adjacent to the 60-in conduit. When the new filter plant was placed in operation, it was necessary to re-arrange the operation of the coagulating basins and advisable to abandon the method of injecting the alum solutions, as inadequate mixing and other factors were preventing the proper removal of suspended matter.

In 1923, a small temporary wooden building for housing two alum tanks and the dry feed lime machine was constructed at the outlet tower of Raw Water Basin No. 2, and experiments were made in the injection of chemical solutions into the entrance of the 60-in. raw water conduit to the coagulating basins. The results obtained justified the change; but the necessity for a structure to facilitate proper mixing under varying conditions prior to coagulation still exists, as occasionally "red" and "muddy" water is produced by the filtration plant.

After careful investigation and study, it was decided to install a gravity-operated mixing chamber, between the raw water and coagulating basins, with a chemical house superimposed over the inlet to the mixing chamber. The design contemplates a structure built in three sections, which can be operated at rates of 30 000 000, 45 000 000 and 60 000 000 gal. per day, respectively, with the detention time element ranging from 7 to 45 min. The length of travel
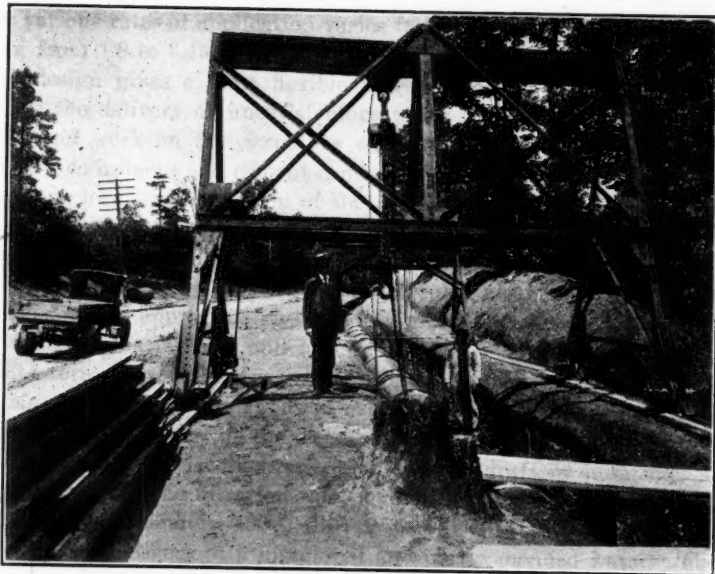
Fig. 5.—View of Pipe Gantry for Laying 48-Inch Main, Lead Gantry in Background, Atlanta, Ga.

Fig. 6.—View of Differential Surge Tank; Raw Water Mains, Atlanta, Ga.

FIG. — VIEW OF FIRE GIRDER AND LIVING TRUSS OF HIGH-LEVEL RESERVOIR IN HEMPHILL AVENUE, ATLANTA, GA.

FIG. — VIEW OF DISTRIBUTING STAND TANK, NEW WATER WORKS, ATLANTA, GA.

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for the various rates of application varies from 1 200 to 3 000 ft., with velocities ranging from 1.8 to 3.45 ft. per sec.

The design gives a most flexible and satisfactory arrangement and will eliminate the delivery of unsatisfactory water to the filters. Bids have been received and work on this structure which is to be reinforced concrete with cast-iron pipe influent and effluent conduits, is to be started as soon as the steps necessary to legal condemnation of the necessary land area are completed.

New Filter Plant and Clear Water Reservoir.—The pressure type filters of the existing filter plant having a combined rated capacity of 21 000 000 gal. per day, discharged into two open circular clear water wells of a combined capacity of approximately 2 000 000 gal. The elevation of water in the coagulating basins is 985.0 and that of the water surface in the clear wells, 945; therefore, 40 ft. of head was lost in operation.

Because of the daily demands in excess of the normal rating of the filters and the limited capacity of the clear wells, it was necessary to vary the rate of filtration to meet the fluctuations in rate of draft or pumpage, and this constant change of the filtration rate, together with the overload on the filters which was approximately 100% during periods of peak demand, was often reflected in the delivery of a discolored and partly purified water. The maximum rate of draft had reached 40 000 000 gal. per day in 1920, and the maximum rate of filtration through the pressure filters had reached approximately 250 000 000 gal. per acre per day. It became imperative, therefore, to construct additional filters and a clear well of sufficient capacity to permit a more uniform filtration rate.

It was decided to construct a covered clear water basin of 10 000 000 gal. capacity, and a gravity filter plant of 21 000 000 gal. daily capacity designed for future expansion to 60 000 000 gal. per day. Construction was started March, 1922, and the new plant was placed in operation in July, 1923.

The water surface of the new filters was at Elevation 977, or 8 ft. below the high-water elevation of the coagulating basins, in order to provide for loss of head in operating and a draw-down capacity of raw water and coagulating basins for future pre-filtration processes and emergencies.

The high-water elevation of the new clear water basin was at 967, or 10 ft. below the water surface in the filters. This elevation was 22 ft. higher than that of the water surface in the old clear water wells and necessitated the abandonment of these wells. The total loss in head through the filter plant now became 18 ft., and a saving of 22 ft. of head was obtained.

This higher elevation did not, however, render the pressure type filters inoperative, because a differential of 18 ft. with full clear water basin still existed, and this head is more than enough to operate them at their normal rate.

The filter plant (Figs. 7 to 11) consists of seven 3 000 000-gal. daily units, arranged in a straight line with an open pipe gallery. They are of reinforced concrete, 28 by 42 ft. in plan, have a net sand area of 1 040 sq. ft., or 0.0239 acre each, and are designed to operate at the rate of 125 000 000 gal. per acre per day. Each filter is equipped with twelve concrete wash-water troughs, 7 ft. center to center, which discharge into a central drain dividing the filter into
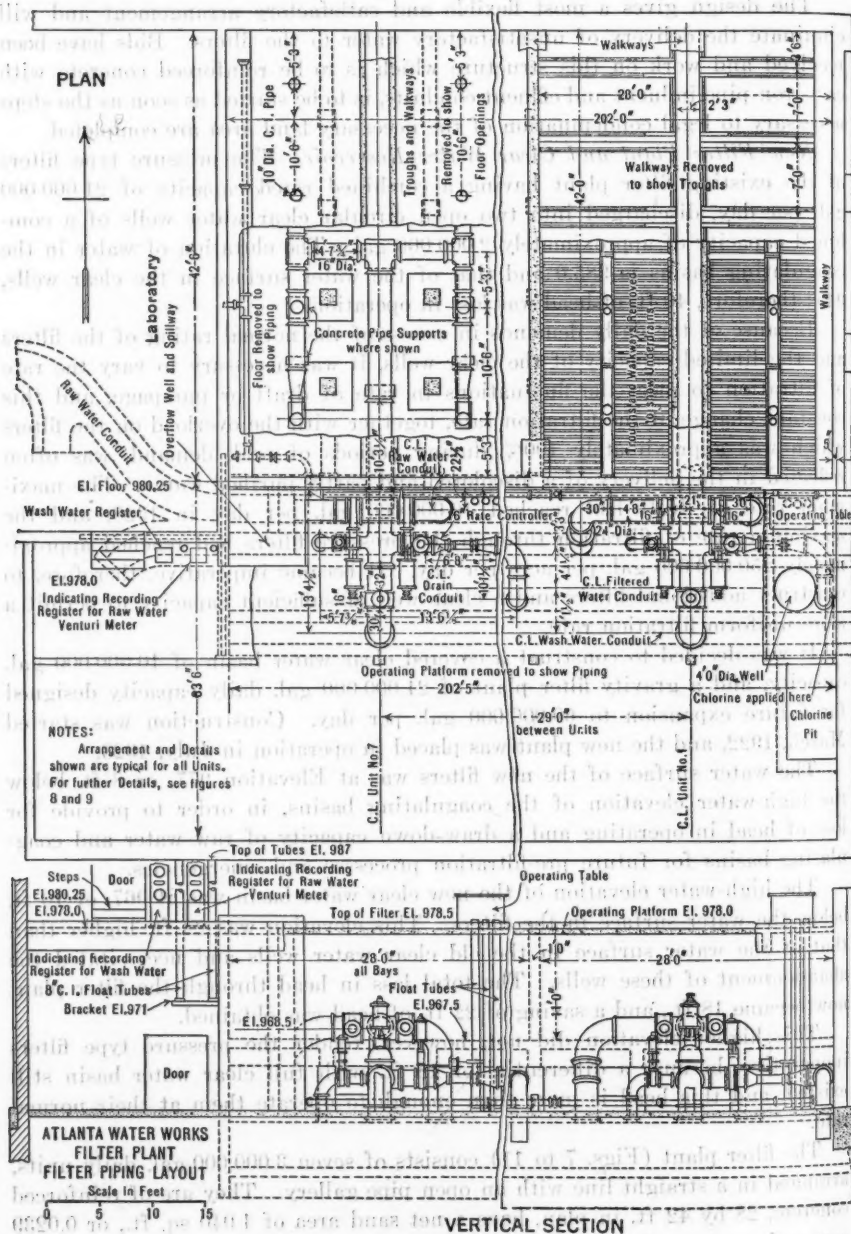
PLAN

Laboratory

Walkways

10' Dia. C. I. Pipe

Troughs and Walkways

Removed to show

Floor Openings

28'-0"

202'-0"

2'-3"

Walkways Removed to show Troughs

42'-0"

Troughs and Walkways Removed to show Underdrains

Walkway

Floor Removed to show Piping

16" Dia.

Concrete Pipe Supports where shown

C. I. Raw Water Conduit

Raw Water Conduit

Overflow Well and Spillway

El. Floor 980.25

Wash Water Register

30" Rate Controller

6"

30"

4'-8"

Operating Table

24" Dia.

El. 978.0

Indicating Recording Register for Raw Water Venturi Meter

C. I. Drain Conduit

C. I. Filtered Water Conduit

C. L. Wash Water Conduit

Operating Platform removed to show Piping

202'-5"

C. L. Unit No. 7

29'-0" between Units

C. L. Unit No. 1

4'-0" Dia. Well

Chlorine applied here

Chlorine Pit

63'-6"

NOTES:
Arrangement and Details shown are typical for all Units. For further Details, see figures 8 and 9

Top of Tubes El. 987

Steps Door
El. 980.25
El. 978.0

Indicating Recording Register for Raw Water Venturi Meter

Top of Filter El. 978.5

Operating Table

Operating Platform El. 978.0

Indicating Recording Register for Wash Water

8" C. I. Float Tubes

Bracket El. 971

28'-0" all Bays

Float Tubes El. 967.5

El. 968.5

Door

1'-0"

28'-0"

ATLANTA WATER WORKS FILTER PLANT FILTER PIPING LAYOUT

Scale in Feet
0 5 10 15

VERTICAL SECTION

FIG. 7.

ATLANTA WATER WORKS FILTER PLANT — CROSS SECTION OF PLANT, DETAILS OF MANIFOLDS AND LATERALS

CROSS SECTION THROUGH FILTER LOOKING WEST

LONGITUDINAL SECTION THROUGH FILTERS

SECTION OF MANIFOLD

SECTION OF LATERALS

ELEVATION OF CHAIR

PLAN OF COLLECTING SYSTEM

SECTION OF MANIFOLD SHOWING PIPING

END ELEVATION OF MANIFOLD

ELEVATON OF MANIFOLD

PLAN OF MANIFOLD

FIG. 8.

ATLANTA WATER WORKS FILTER PLANT STRUCTURAL DETAILS

FIG. 9.

FIG. 10.—WASH-WATER TROUGHS, NEW FILTERS, ATLANTA, GA.

FIG. 11.—INFLUENT, EFFLUENT, WASH AND DRAINAGE CONNECTIONS,
NEW FILTRATION PLANT, ATLANTA, GA.

FIG. 10.—Wash-Water Troughs, New Filters, Atlanta, Ga.

FIG. 11.—Influent, Effluent, Wash and Drainage Connections,
New Filtration Plant, Atlanta, Ga.

two parts. The walls of this drain are 6 ft. high and carry a concrete walkway which extends from the operating platform to the rear of the filter. (See Figs. 8 and 10.)

The under-drain system consists of a cast-iron manifold, 10 in. in diameter, divided into eight sections and having eight 10-in. outlets to the effluent main. The laterals are of 2½-in. cast-iron pipe spaced 6 in. center to center. Laterals are drilled in the bottom quadrant with ⅛-in. holes at an angle of 45°, center to center, and staggered. (See Fig. 8.)

The sand bed is 30 in. deep, and the sand has an effective size of 0.36 mm. and a uniformity coefficient of 1.5. The gravel supporting this sand is placed in five layers and is 20 in. deep.

The filters are entirely covered with a brick superstructure, 76 by 202 ft. At the west end of the building (Fig. 7) is located a head-house, laboratory, and office, 60 by 76 ft., which eventually will occupy a central position when the plant reaches its ultimate capacity.

The coagulated water enters at the west end of the plant through a 48-in. cast-iron main and Venturi meter. The level of water on the filter beds is controlled by a 42-in. hydraulic valve set in this line, which is operated by a float and pilot-valve mechanism set on the operating platform in an open chamber, 3 by 4 ft., connected with the coagulated water conduit. This chamber is also equipped with an emergency overflow weir and is connected with the main drain conduit to prevent the overflow of the filters if the control valve fails to operate.

The four main conduits in the filter building below the filter gallery are of reinforced concrete, side by side, and were poured monolithic in sections in one operation. These conduits were designed for an ultimate capacity of 60 000 000 gal. per day. The tops of the conduits are level with and form a part of the lower floor of the building. The bottom of the filters is about 5 ft. above this floor and is supported on concrete columns.

All collector piping beneath the filters is exposed and easily accessible. The under-side of the filters is shut off from the main part of the building by a 6-in. concrete curtain-wall, but a door is provided for each filter.

All operating piping, hydraulic valves, controllers, etc., are exposed to view from the main floor and are arranged so that all are readily accessible for repair or adjustment. This arrangement is perhaps somewhat greater in first cost than the usual pipe gallery with filters on both sides, but it certainly results in more efficient operation and permits inspection and ease of maintenance because of its accessibility. The operating platform is 15 ft. wide, and is supported above the conduits by concrete columns. It is reached from the main floor by a concrete stairway in the center, and by circular steel stairways at both ends of the building. Fig. 11 is a view of the influent, effluent, wash, and drainage connections.

Wash water is supplied to the filters by a reinforced concrete wash-water tank of 425 000 gal. capacity, located about 50 ft. north of the filter building, with the normal water surface at Elevation 1 000, or 23 ft. above the water surface in the filters. Filtered water is supplied to this tank by two 4 000-gal. per min. motor-driven centrifugal pumps arranged in duplicate. The pumps

take suction from a well connected to the main effluent conduit and are auto-matically controlled by float switches in the tank and suction well. The pump intake is arranged so that the water in the effluent conduit cannot be drawn down below the outlets of the filters, which would break their seals.

The tank discharges into the wash-water conduit through a 30-in. cast-iron main in which is set a 30-in. Builders Iron Foundry controller. The register for this controller is mounted on the operating platform and indicates the rise of wash water in the filters, in inches per minute and in gallons per minute, and records the time, duration, and rate of wash. A change in the rate of wash can be quickly and easily made by adjusting the instrument. It also prevents a sudden rush of water due to quick opening of the hydraulic valve on the wash-water main, because the normal position of the controller is closed, and it requires at least 1 min. to open it.

The effluent from the filters enters the filtered water conduit through a 16-in. cast-iron line in which is set a 16-in. Builders Iron Foundry controller. The effluent conduit is sealed by a weir to prevent the lowering of the water below the filter outlets when the clear-water reservoir is drawn down below the effluent conduit.

At the east end of the filter building, chlorine is injected into the filtered water, and the water then passes through a 60-in. cast-iron main through a Venturi meter and a controlling gate-house to the clear water basin, or the pumps, the clear water reservoir floating on the line.

In the gate-house are located the various valves for controlling the flow from the clear water well and the filters. The 60-in. line is here reduced to 48 in. in order to decrease the valve cost and make operation easier. All valves have been arranged for electrical operation in the event this becomes necessary or advisable in the future.

The pressure filters have been connected to the clear water basin, and the effluent line from the new filter plant, and the supply line between the gate-house and the high-service pumps. Provision has also been made at the valve-house for an additional suction main to be laid in the future. When this is constructed, there will be two 48-in. and one 36-in. suction lines to the high-service pumping station.

The operation of the pressure filter plant in parallel with the gravity plant is accomplished by connecting the effluent from the pressure plant to the con-duit conducting the water from the new clear water basin and gravity plant to the high-pressure pumping station, by a 36-in. cast-iron main. The pressure filters are located at Elevation 945, and the water surface of the clear water reservoir is at Elevation 967. To prevent back pressure on the pressure filters when the clear water basin is full, a check-valve has been inserted in the 36-in. line. The pressure filters must build up sufficient head to open this check-valve against the pressure from the new clear water basin, or the pressure from the gravity plant, before the filtered water from the pressure plant can pass into the system. With a full clear water basin a maximum differential of 18 ft. exists between the influent and the effluent, and this head has been found sufficient to overcome the losses due to the filters and pipe friction and

to permit the operation of the pressure filters at their normal capacity in parallel with the gravity plant.

The old filter plant has no loss-of-head gauges, or rate-of-flow controllers, and the filters are washed by coagulated water. Detail plans for these improvements, including connection to the new wash-water tank of the new filter plant, have been prepared, and bids received, but contracts have not yet been awarded. This improvement is most necessary, and will probably be carried out during 1925.

Clear Water Reservoir.—The clear water reservoir has a capacity of 10 000 000 gal. It is trapezoidal in plan, 215 by 388 ft. at the top with 2 to 1 slopes, 22 ft. deep, constructed entirely in cut, and has a reinforced concrete roof of slab-and-beam construction which is covered with 18 in. of earth.

Small entry houses are provided at both ends of the structure, and concrete stairs extend along the slopes from the entry houses to the bottom of the reservoir. Fig. 12 is a view of the interior of the new clear water reservoir.

The floors and slopes are of concrete 6 in. thick and reinforced with electrically welded steel fabric of No. 4 wire, 4 in., center to center. In order to insure water-tightness, concrete beams or sills, 12 by 12 in., were first poured on the side slopes, 18 ft., center to center, with the tops of the beams level with the under-side of the slab. Into these beams, at intervals of 6 ft., were inserted bolts to hold forms. Bulkheads were erected on the center line of beams, and the slabs were then poured with a moderately wet mix, the forms being carried up 18 in. in advance of the concrete. Three slabs were usually poured simultaneously, and in this way the pressure on the forms was kept within reasonable limits. At the joints in the slabs over the beams, No. 18 gauge, galvanized-iron strips were inserted, extending 4 in. into the adjoining sections. Before the adjoining section was poured, the joint was covered with ½ in. of hot pitch.

After completion, tests extending over a period of 20 days indicated a drop in water level with full reservoir of $\frac{3}{32}$ in. per 24 hours. Some of this leakage was traced to the 48-in. valve in the discharge line from the reservoir.

Water enters and leaves the basin at the south end through a 60-in. cast-iron pipe laid under the floor, which is later reduced to 48 and 36 in. From this line are taken three sets of 24-in. cast-iron laterals which extend at right angles to the main conduit and discharge at the foot of the side slopes. These laterals are fitted with 90° bends and extend about 2 ft. above the floor of the basin. The bends on each pair of laterals are turned in opposite directions to facilitate circulation.

The clear water basin acts as a balancing unit for the filters, and the filter plant is now being operated at a uniform rate of approximately 26 000 000 to 30 000 000 gal. daily, the new plant supplying 21 000 000 gal. and the pressure plant the difference.

OPERATING RESULTS

The new filter plant (Fig. 13) has been in operation since June, 1923, and the results produced from both the bacterial and the operating standpoint have

been excellent, except when the water delivered to the filter plant was overloaded with suspended matter. The operation of the filters at a uniform rate has reduced some of these troubles, and, although the pre-filtration processes are seriously in need of improvement, the entire plant is now functioning much more satisfactorily than it did under a constantly changing rate.

There has been no evidence as yet of the formation of mud balls, or cracking or shrinkage of filter beds, and the distribution of wash water is quite uniform, which indicates that the design of the under-drain system is satisfactory.

Table 4 is a daily operating log of the plant for the six-month period ending January 31, 1924. These data indicate unusually good results, but inasmuch as they are the combined record of both the old pressure type and new gravity filter plant, they do not show some of the more interesting records of the operation of the latter plant.

The average length of run between washings of the gravity units varies from 60 to 70 hours, with maximums of more than 120 hours. The quantity of wash water required for these has been less than 1 per cent.

The open type filter gallery with its ease of inspection, operation, and maintenance, justifies this type of design.

The new plant is provided with every facility and is heated from the boiler plant at the high-service station. Ample radiation produces satisfactory temperature during zero weather, which occurs very rarely.

Improvements at Hemphill Avenue Pumping Station (High Pressure)

The improvements at the Hemphill Avenue Station consisted of a new radial brick chimney with a steel flue for future extensions; remodeling the steam headers; equipping the boilers with superheaters and mechanical stokers; installing a de-superheater to provide saturated steam for certain units; remodeling of suction and discharge headers; removing one of the old 10 000 000-gal. pumps; and installing one 30 000 000-gal. turbo-centrifugal DeLaval pumping unit, having practically the same guaranty as those at the Chattahoochee Station.

As this station pumps directly into the distribution system with no equalizing reservoir, the operating conditions are variable, the domestic demand ranging from a rate of 17 000 000 to 18 000 000 gal. per day at night to a rate of 45 000 000 to 50 000 000 gal. per day peak rate during daylight hours, and varying with the season of the year.

The construction of the new clear water reservoir (high-water elevation, 967) has placed all pump suctions under a positive head, which reduces the differential between water surface and maximum head, thus materially reducing unit pumping costs.

The average domestic pressure maintained at this station ranges from 80 to 100 lb. It became necessary to build this pressure up from 120 to 125 lb. during fire-drafts, due to inadequate feeders and supply mains to the distribution system. The unprecedented growth of the City of Atlanta and its suburban area, with rapidly increasing demands, emphasizes the necessity of a prompt survey of the distribution system and an analysis of flows, pressures, and drafts to determine the reinforcement necessary.

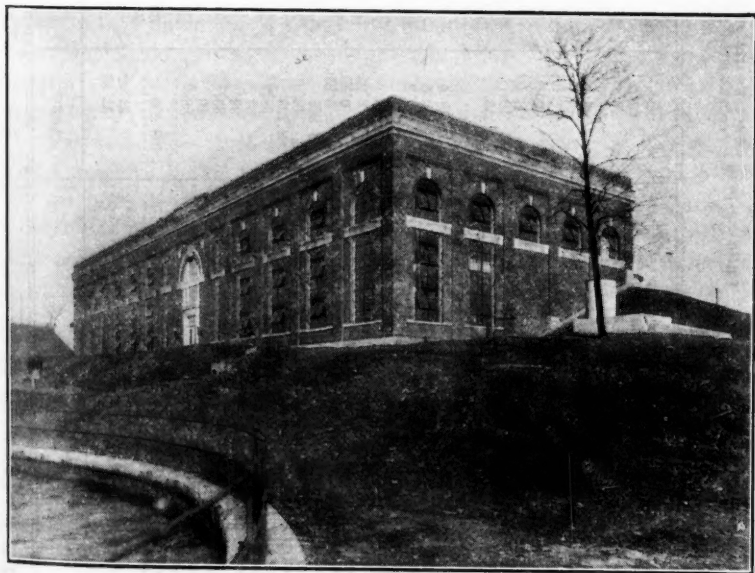FIG. 12.—VIEW OF INTERIOR OF NEW 10 000 000-GALLON CLEAR WATER RESERVOIR, ATLANTA, GA.

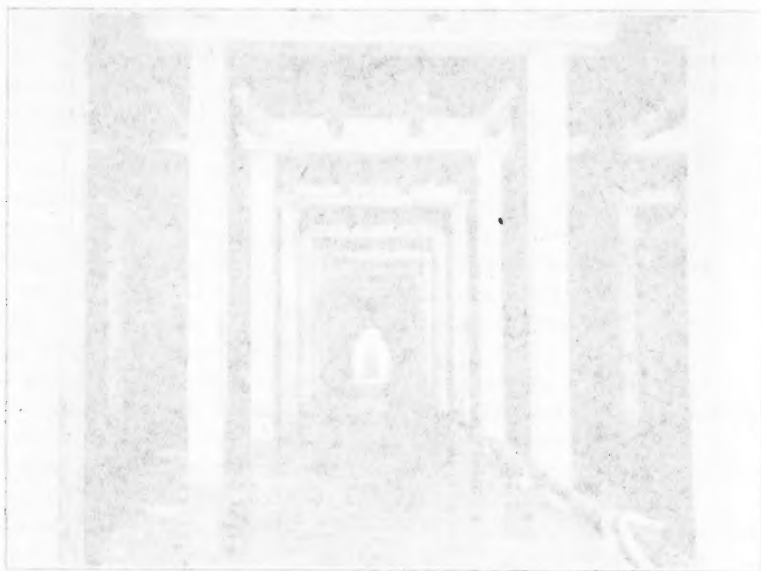FIG. 13.—VIEW OF NEW GRAVITY FILTRATION PLANT, ATLANTA, GA.

FIG. 13.—VIEW OF INTERIOR OF NEW SEDIMENTATION BASIN PUMP HOUSE, ATLANTA, GA.

FIG. 12.—VIEW OF NEW GRAVITY FILTRATION PLANT, ATLANTA, GA.

TABLE 4.—SUMMARY OF OPERATING RESULTS, ATLANTA WATER-WORKS PURIFICATION PLANT, JUNE TO DECEMBER, 1923.

	June.	July.	August.	September.	October.	November.	December.
Million gallons coagulated, daily	27.89	28.94	28.76	27.08	26.05	25.59	25.36
" filtered daily	27.45	28.71	28.25	26.62	25.50	25.21	25.00
Wash water, gravity filters, in gallons per day	408 600	571 416	400 846	356 296	378 965	335 833†	361 429‡
Wash water, pressure filters, in gallons per day		230 022	181 900	178 867	149 179	186 915	123 442
Percentage of wash water to total	2.8	2.8	1.8	1.8	1.9	1.6	1.4
Maximum run, gravity filters, in hours	68¾	76½	98¼	122½	98	99¼	98
Minimum run, gravity filters, in hours	10	8	10½	66¼	40¾	50¼	51¼
Average run, gravity filters, in hours	39¼	39	45¼	94¾	68¾	70	73
Maximum run, pressure filters, in hours	36	44	37	30	26	24	24
Minimum run, pressure filters, in hours	24	18	14	24	24	17	24
Average run, pressure filters, in hours	28¾	28¾	27½	27	24	23¾	34
Alum used, pounds per day	3 132	3 954	5 952	2 888	1 428	1 296	1 359
Chlorine used, pounds per day	48.6	49¼	44.9	40.6	42.7	41.0	40.0
Lime used, pounds per day	1 865	1 802	2 863	1 543*			
Alum used, in grains per gallon	0.89	0.96	1.04	0.80	0.38	0.35	0.37
Chlorine used, in parts per million	0.22	0.22	0.20	0.18	0.20	0.20	0.20
Lime used, in grains per gallon	0.57	0.44	0.60	0.39*			
River water turbidity	309	412	768	252	92.5	720	161
Raw water turbidity	242	274	395	234	63.6	87	59.9
Coagulated water turbidity	34.5	37.3	42	29.5	24	24	25
Alkalinity, raw water	13.9	15.8	15	15	21.8	23.6	19
Alkalinity, coagulated water	11.6	15.5	15.5	12.9	17.5	21.5	17
Bacteria per cubic centimeter on agar at 37½° cent.:							
Raw water	772	1 142	514	565	271.1	163	186
Coagulated water	79	128	85	29	31.4	17	26
Filtered water	6.1	49	28	5.1	9.1	5.1	3.7
Tap water	5.1	7	0.9	1.5	3.1	1.4	1.7
Bacillus coli:							
Raw, in 1 cu. cm., percentage positive	60	45½	25.8	16.7	3.2	3½	25.8
Raw, in 0.5 cu. cm. percentage positive§	94.9	80.7	58.1	41.7	11.8	28.3	72.6
Filtered, in 10 cu. cm., percentage positive‖	0	13.0	0	0	0	8.3	11.3
Tap, in 10 cu. cm., percentage positive¶	8.3	0	0	0	0	0	0
Tap, in 100 cu. cm., percentage positive¶	0	0	0	0	0	0	0

Gravity. Pressure.
Average time, in minutes .... 6.26 .... 17.9 } Filter washing
Average rise, in inches per minute. 24.7 .... 11.2 }
Maximum for any washing, gravity, 26.5 in. " " 19.7 in. } Maximum / Minimum

* Lime applied for 14 days only.
† 24 days only.
‡ 28 days only.
§ Daily average of two samples.
‖ Daily average of two samples.
¶ Daily average of three samples.

Cost of the Work

The following figures indicating the cost of the major items of the work completed, authorized, and contemplated, may be of interest:

Estimated cost, impounding dam, intake, etc........	$2 500 000
Mixing chamber and chemical house................	176 000
48-in. raw water main and connections, including gate-house at reservoirs, aqueduct, tunnel, etc..........	682 000
Filter plant, reservoirs and connections.............	699 000
Three 30 000 000-gal. turbo-centrifugal pumps erected (not including foundations).....................	192 000
Pumping station improvements, including steam improvements, and pump foundations, connections, cranes, etc.	300 000
Equipping thirty-six 500 000 gal. per day horizontal type filters, with loss-of-head gauges, rate controllers, and filtered water wash line with controller........	42 000

General

The entire work has been under the general direction of W. Zode Smith, General Manager of the Atlanta Water-Works.

The 48-in. raw-water main and connections, all suction and discharge headers, pump foundations, connections and exterior piping at the filter plant, were constructed and installed by the forces of the Water-Works Department with William M. Rapp, Superintendent of Construction, in charge. This work has been carried on efficiently and expeditiously, and well within estimates, and at a considerable saving over competitive bids.

The writer, acting as Consulting and Designing Engineer for the City of Atlanta, desires to give full credit and honor to his able corps of designers, resident engineers, and assistants, who have served so conscientiously and efficiently in the preparation of plans and the supervision of construction, including Frank J. Keis, M. Am. Soc. C. E., and Herman F. Wiedeman, Assoc. M. Am. Soc. C. E.; and Messrs. T. M. Sullivan, McDonald Lawrence, M. T. Singleton, Lowell Cady, S. E. Arnold, and others. Mr. Wiedeman was Resident Engineer on the filter plant and clear water reservoir and during the period of "tuning up". Mr. J. N. Eley has co-operated with the writer in all mechanical and electrical matters, and supervised all duty and efficiency tests on the pumps and steam equipment.

HIGH SPECIFIC SPEED HYDRAULIC TURBINES IN THEIR BEARING ON THE PROPORTIONING OF THE NUMBER OF UNITS IN LOW-HEAD HYDRO-ELECTRIC PLANTS

A SYMPOSIUM*

HIGH SPECIFIC SPEED TURBINES

By George A. Orrok,* M. Am. Soc. C. E.

Among the many things of interest seen and heard at the World Power Conference (London, 1924), none perhaps was more striking than the variety of methods proposed and tried for the solution of the important problem of high specific speed turbines. From talks with experimenters and visits to European testing laboratories, from the papers on these laboratories and their work presented to the Conference, and from current literature in many languages, the speaker believes that although the subject is not new and is well known in America, yet the many excellent examples of good design and construction at present under way in Europe should be better known to members.

Among the larger plants, the Chancy-Pougny installation of five wheels (7 600 h.p.), on the Rhone, below Geneva, Switzerland, is most interesting. The turbines are 18 ft. 0 in. in diameter, under 26.7-ft. head, with a characteristic speed of about 120, just a trifle larger than those at Keokuk, Iowa. The wheels are a modified Francis type, retaining the outer rim, and were built by the Des Charmilles Company, of Geneva; they are guaranteed for an efficiency of more than 90% at the best gate-opening.

In comparison, the Lawaczeck wheels for Lilla-Edet, Sweden (Fig. 1), are 19 ft. 8 in. in diameter without the outer rim, and have a characteristic speed of 136; the head is 21.3 ft. instead of 26.7 ft., the power is 10 000 h.p., and the water used is 5 200 cu. ft. per sec., instead of 3 250 cu. ft. per sec. A similar wheel, 9 ft. 6 in. in diameter, at the Viereth plant in Germany gives 2 000 h.p., with a 17.4-ft. head and a speed of 93 (Figs. 2 and 3). This latter wheel was made by the Neumeyer Company, of Munich, Germany, and showed test results considerably better than runners of the Francis type for the same head and speed. About sixty runners of this type have been tested during the last year (1924).

The Lilla-Edet plant has two of the 19 ft. 8-in. Lawaczeck runners installed, and the third unit is a propeller runner with adjustable blades of the Kaplan type. This third runner has about the same diameter and water capacity as the others, but the blades are adjustable, giving much better efficiency at low gate-openings. The Swedish Government has installed a testing laboratory at Lilla Edet and models of both wheels, 1 m. in size, were given an exhaustive test before the contracts were signed. Dr. D. Thoma, Professor of Hydraulic Engineering, Technische Hochschule, Munich, Germany, has stated that the form of the Lawaczeck runners is a mathematical surface with three parameters and that the test efficiencies may be related to the equations of the surface. He further states that this method of analysis will prove fruitful without restriction to any special theory of turbine design.

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* Cons. Engr., New York, N. Y.
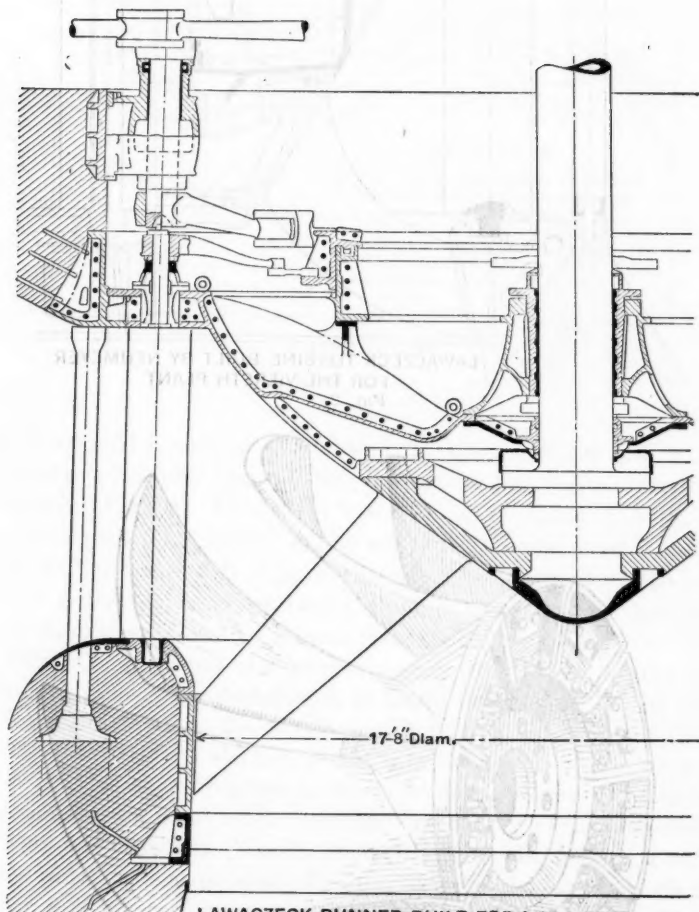
LAWACZECK RUNNER BUILT FOR LILLA EDET

Fig. 1.
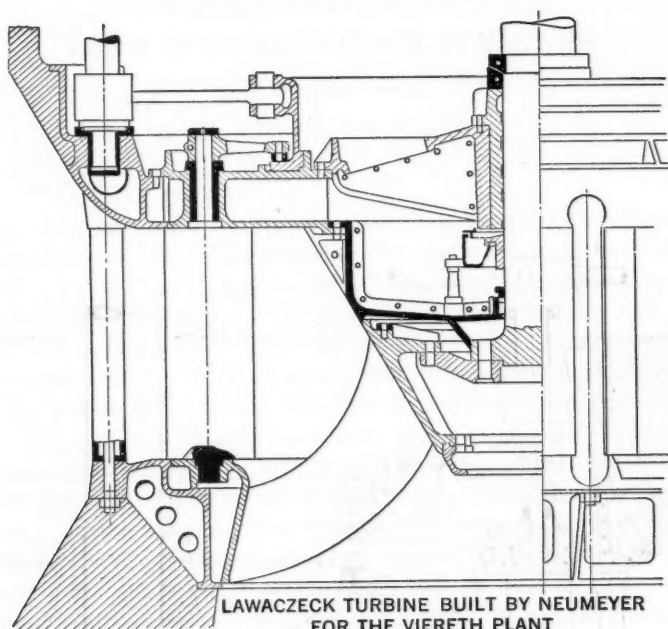
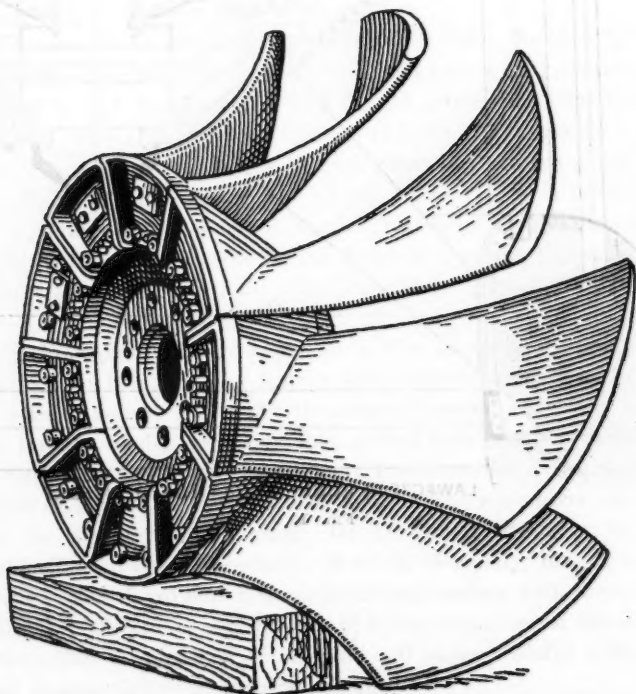LAWACZECK TURBINE BUILT BY NEUMEYER
FOR THE VIERETH PLANT
FIG. 2.

FIG. 3.—LAWACZECK RUNNER (2 900 MM. IN DIAMETER) FOR
THE VIERETH PLANT.

The Kaplan runner at Lilla Edet is 19 ft. 0 in. in diameter and gives 11 200 h.p. It has four blades, adjustable while running. The design included a special draft-tube for each type of wheel. Actually, however, the second Lawaczeck runner was installed in a draft-tube designed for the Kaplan runner. The performance of the wheels at Lilla Edet is shown in Fig. 4.

COMPARISON OF
KAPLAN AND LAWACZECK TURBINES
AT LILLA EDET POWER STATION

Calculated Efficiencies when enlarging Diameters of Runners to 19 ft. and 19 ft. 8 in. respectively.
Guaranteed Efficiencies.
Efficiencies of the Wheel Models braked at Lilla Edet.
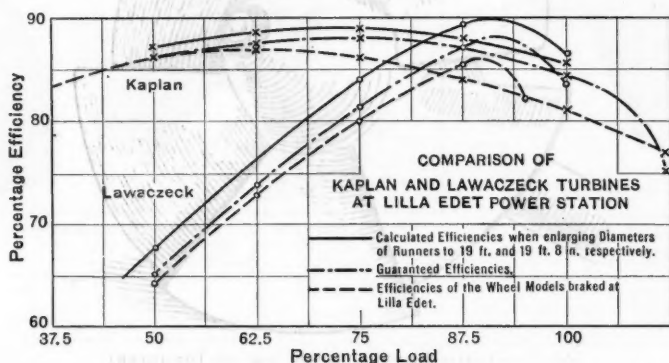
Fig. 4.

J. M. Voith and Company, of Heidenheim, Germany, has also manufactured a number of Kaplan runners, the first having been installed at Siebenbrunn, Austria (Fig. 6). This wheel is 6 ft. 3 in. in diameter, with 20.3-ft. head, and develops 400 h.p. at 250 rev. per min. The specific speed is about 212. There is a similar wheel, 5 ft. 1 in. in diameter, with a speed of 250 rev. per min., under a 13.75-ft. head, at Gävle, Sweden; apparently, a considerable number of these wheels have been made by other manufacturers.

The Voith Company has also been manufacturing wheels of the Kaplan type with fixed vanes. The installation at Kachlet, Germany, on the Danube, is quite interesting. Here, the wheels are 15 ft. 1 in. in diameter (Fig. 5), under a 25.1-ft. head, for 7 450 h.p., giving a specific speed of about 115. The runner has six blades covering nearly the entire area, whereas the four-bladed Kaplan designs do not cover much more than one-quarter of the projected area. Escher, Wyss and Company (Zurich, Switzerland), is also making a propeller turbine for which tests show good efficiency. In Fig. 7 is shown a six-bladed model; Fig. 9 shows performance curves of a four-bladed model. The tests of an interesting two-bladed helical runner (Figs. 8 and 10), made by Theo. Bell and Company (Kriens, Switzerland), have been published by Dr. F. Prazil, in Zurich. A turbine of this type for which the performance curve is shown in Fig. 10, is situated at the Matte Central Station in Berne, Switzerland. Dr. Hahn has given a table of test efficiencies of runners of this type collected from a number of European sources. Statistics of some of these high-speed European plants are given in Table 1 and comparative performances in Fig. 11.
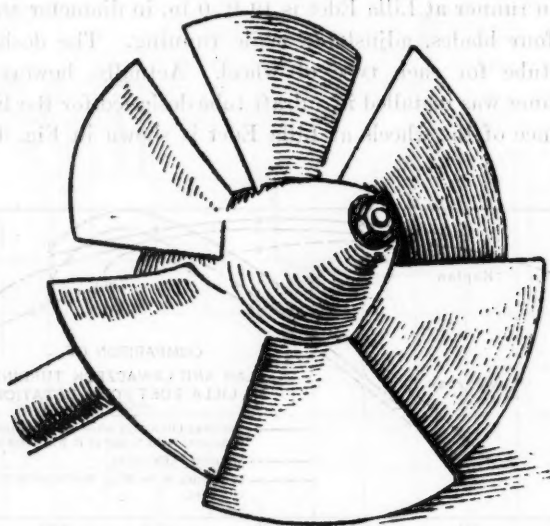
FIG. 5.—VOITH RUNNER (4 600 MM. IN DIAMETER)
FOR THE KACHLET PLANT.

In his paper before the World Power Conference, Dr. Thoma emphasizes the fact that the tests on small-sized, geometrically similar models give reliable figures for large-sized operations and that in the few cases of divergence noted a lack of geometrical similarity has been proven. He also states that the Camerer correction formula for efficiency has been found to be fairly correct. He gives a list of thirteen hydraulic testing laboratories actually running in Germany alone.

TABLE 1.—EUROPEAN LOW-HEAD TURBINES.

Manufacturer.		Location.	Diameter.	Discharge, in cubic feet per second.	Head, in feet.	Revolutions per minute.	Horse-power.	Specific speed.
Charmilles, S. A...	F	Chancy-Pougny, Switzerland	18 ft. 0 in.	3 250	26.7	83.3	7 600	± 120
A. B. Finshyttan*	L	Lilla Edet, Sweden	19 ft. 8 in.	5 200	21.3	62.5	10 000	± 135
A. B. Finshyttan†	K	Lilla Edet, Sweden	19 ft. 0 in.	5 300	21.3	62.5	11 200	± 140
Voith	K	Siebenbrunn, Austria	6 ft. 3 in.	.....	20.3	250	400	212
A. B. Karlstads	K	Gavle, Sweden	5 ft. 1 in.	.....	13.75	250	400	219
Voith	H	Kachlet, Germany	15 ft. 1 in.	.....	25.1	.....	7 450	115
Neumeyer	L	Viereth, Germany	9 ft. 6 in.	.....	17.4	.....	2 000	93
Vevey	H	Wynau, Germany	10 ft. 10 in.	1 700	17.0	107	2 700	165

* Lawaczeck wheel.
† Kaplan wheel.

It is fair to state that Dr. Thoma emphasized the cavitation difficulty and is testing models under cavitation conditions. Half his paper was devoted to
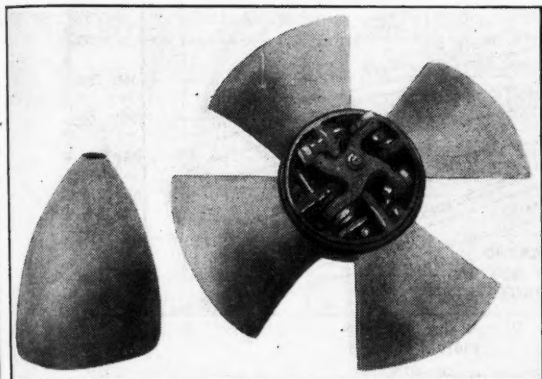
FIG. 6.—KAPLAN RUNNER AT SIEBENBRUNN PLANT,
SHOWING MECHANISM FOR MOVING
PIVOTED BLADES.
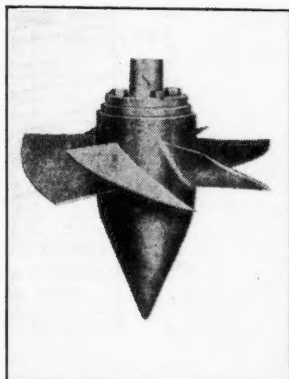
FIG. 7.—DUBS PROPELLER RUNNER
MADE BY ESCHER WYSS
AND COMPANY.
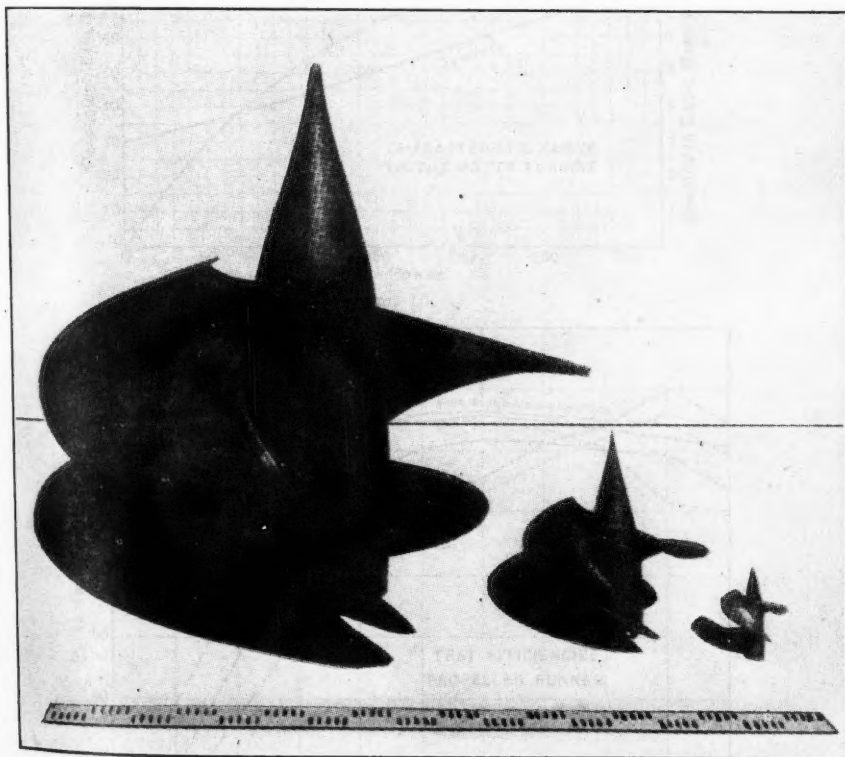
FIG. 8.—BELL RUNNER FOR THE MATTE PLANT, WITH THE TWO SMALL MODELS
USED IN THE TESTS.

CHARACTERISTIC CURVES
FOR THE DUBS TURBINE
FOUR BLADED MODEL

Fig. 9.

CHARACTERISTIC CURVE
OF THE MATTE TURBINE

Fig. 10.

TEST EFFICIENCIES
PROPELLER RUNNER

FROM L'ENERGIA ELETTRICA
OCT., 1924

Fig. 11.

this subject. In conversation, he stated that this problem in the high-speed runner can be solved. Meanwhile, reports as to the performance and life of these large sized and particularly high-speed runners will be awaited with avidity. The speaker believes the high-speed runner has come to stay and that present difficulties with this type will be overcome, as with the older and better understood designs.
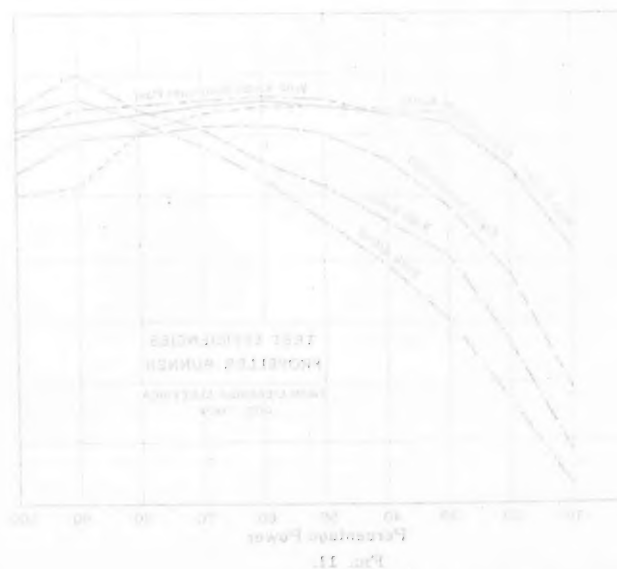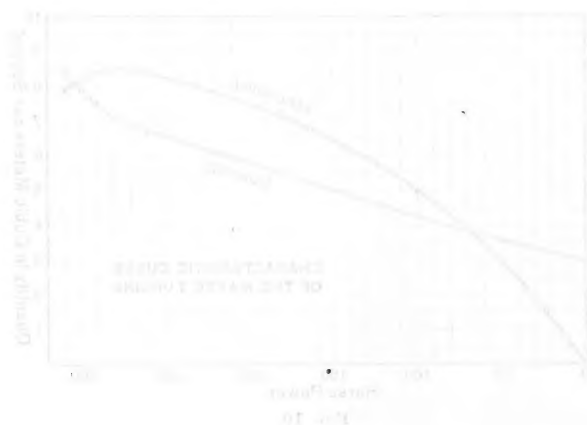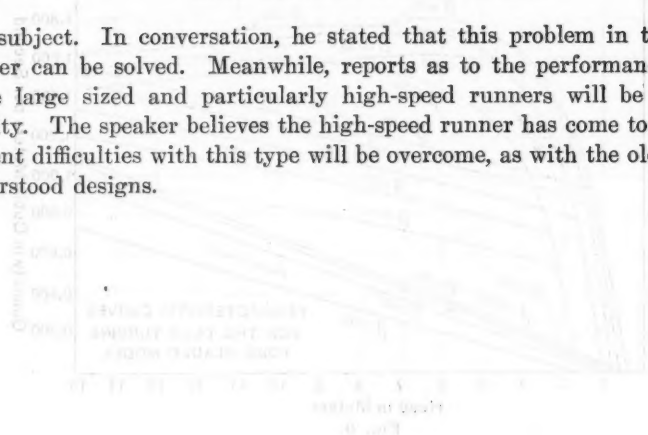
THE PROPELLER TYPE TURBINE

By Lewis F. Moody,* Esq.

To any one who has followed hydraulic power engineering it must seem surprising that a machine so highly developed as was the hydraulic turbine ten or fifteen years ago, should have suddenly entered a period of radical changes and encountered transformations in many of its essentials. The fact remains, however, that although practice in turbine design had become almost standardized in the period just prior to the World War, the development of this art has taken on new life and has already been revolutionized in many aspects.

The most striking changes have been those in runner and draft-tube design, resulting in a remarkable increase in the specific speeds. This paper is directed particularly to one form of high-speed turbine and to the description of a number of typical installations, representing some of the most recent developments in this field. Before describing particular installations, however, a few general relations will be discussed leading to some of the reasons which account for the adoption of the propeller type of runner and which explain at least in part the logic of its development.

The first point discussed is not directly pertinent, but is of some interest in itself, namely, the relation between size of turbine and efficiency; the second point then follows—the relation between type or form of turbine and its efficiency and specific speed. The results of several actual tests of models of the propeller type and of large-sized installations will then be presented.

Variation of Efficiency with Size of Turbine

In order to compare results secured on different sizes of models with full-sized turbines, the writer has believed it advisable to take up the question of the effect of size on turbine performance in a series of homologous turbines. In recent years calculations for this purpose have usually been based on the Camerer formula, of the form:

$$e' = 1 - (1 - e) \frac{0.12 + \sqrt{\left(\dfrac{f_2}{U_2 D}\right) D'}}{0.12 + \sqrt{\left(\dfrac{f_2}{U_2 D}\right) D}} \cdot 0.015$$

in which, e equals the efficiency of one turbine; e' that of another turbine; D and D' are their diameters; and $\dfrac{f_2}{U_2 D}$ is the ratio of the outflow area, f_2, of the

* Cons. Engr., I. P. Morris Dept., The William Cramp & Sons Ship & Engine Building Co., Philadelphia, Pa.

runner buckets to the product of the wetted perimeter and the runner diameter; that is, the ratio of the hydraulic radius of the runner bucket exit area to the runner diameter.

Although the Camerer formula has been found to be in fairly good agreement with actual results of tests, it is not altogether satisfactory for the following reasons:

1.—It involves the calculation of the hydraulic radius of the water passages through the turbine and cannot be accurately applied unless the design of a particular turbine is available. It is complicated in form and involves a number of constants or coefficients which can only be accurately determined by applying it to a number of installations of similar general character to any case under consideration. In determining the hydraulic radius to be applied in the formula, it would be necessary to arrive at some average value which would fairly represent the entire course of the water through the turbine, a matter of considerable complication even when the design is fully known. Camerer avoids this complication by using the minimum section which usually occurs at the outlet orifice of the runner buckets. Although the formula has been generally used with this assumption, it no longer remains entirely rational.

2.—The formula is also derived from the assumption that all the losses through a turbine are due to surface resistance or so-called "pipe friction", and discards the consideration, which was recognized by Dr. Camerer, that a part of the losses in a turbine is due to eddies, impact, and velocity head rejected at final discharge. The complicated form of the Camerer expression is due to the fact that it is based on Dr. Biel's formula for pipe friction which is an expression involving the sum of three terms.

Considering the fact that the losses in a turbine are dependent on many factors and involve many complications and that in comparing efficiencies only approximate results at best can be expected, the writer believes that a simple form of expression involving only the leading variables and giving a simple dimensional relation without involving the details of design, can be used with a gain both in convenience of application and in accuracy. For this purpose the writer proposes in place of the Biel formula for pipe friction or other complex form of expression, such as the Kutter formula for channels, the use of an exponential formula, and for this purpose suggests either the recent Forcheimer formula:

$$V = k \, R^{0.7} \, J^{0.5}$$

in which,

V = velocity;
R = hydraulic radius; and
J = slope of hydraulic gradient;

which is equivalent to:

$$J = \frac{V^2}{k^2 \, R^{1.4}}$$

or the still more recent Strickler formula:[*]

$$J = \frac{V^2}{K^2 R^{\frac{4}{3}}} .$$

The Strickler formula is based on rational grounds and has been shown to be in close agreement with a large range of actual measurements. If λ represents the proportional loss of head in a given turbine expressed as a fraction of the total head on the turbine, then $\lambda = 1 - e$, in which e is the turbine efficiency.

If $\lambda = 1 - e$ is the proportional loss of head in one runner, say, a small model, assume that a certain fraction, a, of this loss, that is, $a (1 - e)$, remains unchanged as the runner is increased (homologously) in size; while the remaining portion, $(1 - a) (1 - e)$, is due to surface friction, so that the proportional loss in another, say, a larger, runner, will be:

$$\lambda' = 1 - e' = a (1 - e) + \lambda'_f$$

Now the surface friction in any channel may be calculated by the exponential formula:

$$h_f = f \frac{L}{R^x} V^m$$

in which, except for very smooth pipes of small diameter, m may be taken as closely equal to 2, a value corresponding to cast-iron pipes, and x may be taken between 1.4 (Forcheimer) and 1.167 (Hazen and Williams), or as $1\frac{1}{6}$ (Strickler) (also proposed by Manning, 1890).

Calling $x = 1 + p$, in which $p = \frac{1}{6}$, if the Strickler value is used,

$$h_f = f \frac{L}{R} \frac{V^2}{R^p}$$

Expressed as a fraction of the total head on the turbine,

$$\lambda_f = \frac{h_f}{H} = f \frac{L}{R} \frac{V^2}{R^p H}$$

In homologous runners of all sizes, under any head, when operating at equal gate and corresponding speeds, V^2 varies directly as the head, H, so that $\dfrac{V^2}{H}$ = a constant; and in all homologous runners, $\dfrac{L}{R}$ = a constant; and as R is proportional to D, the runner diameter,

$$\lambda_f = \frac{k}{D^p} = (1 - a) (1 - e);$$

and

$$\lambda'_f = \frac{k}{D'^p}.$$

* "Beitrage zur Frage der Geschwindigkeitsformel und der Rauhigkeitszahlen fur Strome, Kanale und geschlossene Leitungen," by A. Strickler, *Schweizerische Bauzeitung*, June 7, 1924, p. 265.

Hence, putting:

$$k = (1 - a)(1 - e) D^P,$$

we have:

$$\lambda' = (1 - e') = a(1 - e) + (1 - a)(1 - e) \frac{D^P}{D'^P}$$

or,

$$\frac{1 - e'}{1 - e} = a + (1 - a)\left(\frac{D}{D'}\right)^P$$

From the application of this formula to a number of actual comparative tests, it was found that if P is taken as $\frac{1}{3}$, then a, the unvarying portion of the loss, is a comparatively small fraction, approximately $\frac{1}{4}$; and it is also found that the whole loss can be closely approximated for usual values by substituting for $a + (1 - a)\left(\frac{D}{D'}\right)^P$, the simpler expression, $\left(\frac{D}{D'}\right)^n$, in which a slight change in P to n closely compensates for the dropping of the constant term. Moreover, there is basis for believing that a varies with size of runner; in a small runner the vanes are relatively thicker, less sharply pointed and less accurately formed, and have a greater eddy loss at their discharge edges and elsewhere. Hence, an expression of the form:

$$\frac{1 - e'}{1 - e} = \left(\frac{D}{D'}\right)^n$$

should be sufficient for all practical purposes in computing the effect of size on efficiency; and an application to a number of actual tests shows this to be true, with n varying but little from the value of $\frac{1}{4}$.

Therefore, the following formula may be used:

$$e' = 1 - (1 - e)\left(\frac{D}{D'}\right)^n$$

or,

$$e' = 1 - (1 - e)\left(\frac{D}{D'}\right)^{\frac{1}{4}}$$

EFFECT OF CHANGE OF FORM OF TURBINE ON EFFICIENCY AND SPEED

The preceding discussion applies to geometrically similar turbines. Consider, next, the effect of change of form or type of turbine. The particular change which it is desired to consider is that from a so-called Francis turbine to a turbine of the propeller type. This change in type involves the following principal alterations: (a) The elimination of the runner band or shroud ring, allowing the blades to rotate within a stationary wall; (b) the introduction of a large transition space between guide-vanes and runner, in which the water follows a free path and turns from a radial to a partly or completely axial direction; (c) the reduction of blade surface so that the blades either overlap each other only slightly or fail to overlap at all and leave clear openings between them; (d) a reduction in the number of blades; and (e) the use of draft-tubes symmetrical about the turbine axis and capable

of efficiently converting into useful head the velocity head of both the meridian and tangential components of the runner discharge velocity, thus permitting both components to be made relatively large.

The effect of each of these changes is to allow the runner to be operated at a higher speed under a given head, by permitting higher relative velocities to be used and a more inclined direction of the absolute velocity of runner discharge.

The reason for eliminating the band or shroud is simple. With the runner speeds and diameters used, the relative velocity of the water in contact with a moving band would be considerably greater than its absolute velocity along the surface of the stationary wall, so that there is less surface resistance on the inner surface of the band when it is stationary than when it rotates; and by replacing the rotating band by a stationary wall, all the frictional resistance of its outer surface is eliminated.

The effect of the transition space is to reduce the relative velocity between the water and the blades at all sections except at the blade tips. If this space is too much enlarged, however, or if the entrance edges of the blades are carried too near the axis, the relative velocity near the hub begins to increase again, the water at that point then tending to rotate faster than the runner, and, in addition, the surface friction due to the long absolute path along the stationary head cover partly discounts the decrease in blade friction. This consideration is one which has led to the writer's diagonal type of runner, intermediate between a radial and a purely axial flow runner. In a similar way, an undue reduction in blade surface while decreasing the surface resistance, if carried too far results in imperfect guidance of the flow and permits other losses which more than offset the gain. It has been found both from theory and experience that unduly small blade area greatly impairs the stability of operation, giving rise to surging, unstable flow, and loss of efficiency, and materially reduces the field of application of turbines of the propeller type in which it is found.

The effect of reducing the number of blades can be readily investigated along the lines of the analysis given previously for homologous runners. Assume a series of runners in which the same ratio of total blade area to disk area is used, but in some of which there are many blades with short and narrow passages between them, and in others of which there are a few blades with long and wide passages, the blade sections being geometrically similar in all, and having the same ratio of length to width of passages. The runners being unshrouded, the wetted perimeter of a runner channel may be taken as approximately equal to twice the blade length from hub to band plus the hub surface; and if the hub surface is neglected and offset approximately by taking the blade length as if it extended to the axis, the wetted perimeter can be called $2\,r$, in which r is the tip radius of the blades, measured in a conical surface. The area of one runner passage is:

$$\frac{r \times 2\,\pi\,\dfrac{r}{2}\,\sin\beta}{z}$$

in which, β is the blade inclination and z is the number of blades. Hence, the hydraulic radius of a runner passage is:

$$\frac{\pi \, r^2 \sin \beta}{2 \, z \, r} = \frac{\pi \, r \sin \beta}{2 \, z}$$

With the same line of reasoning, the proportion of the total head on a turbine lost in blade friction will vary inversely as the hydraulic radius raised to the nth power, or using the Strickler value of $n = \frac{1}{3}$, the ratio of blade friction losses in two runners can be expressed as:
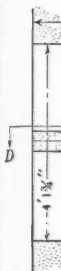
$$\frac{\lambda'_b}{\lambda_b} = \left(\frac{\dfrac{\pi \, r \sin \beta}{2 \, z}}{\dfrac{\pi \, r' \sin \beta}{2 \, z'}} \right)^n = \left(\frac{r}{r'} \, \frac{z'}{z} \right)^{\frac{1}{3}}$$

This relation shows that in such a series of runners as described the blade friction will vary directly as the cube root of the number of blades, and inversely as the cube root of the radius. For example, comparing a 16-vane runner with a 4-vane runner, the latter will have $\left(\dfrac{4}{16}\right)^{\frac{1}{3}} = 0.63$, or 63% of the blade friction loss of the first, or a reduction of 37 per cent. It must be kept in mind that this does not mean keeping the blades the same in size as their number is reduced, but enlarging them to keep the total blade area the same. Although a very small number of blades will be advantageous in reducing this particular loss, there are, of course, certain disadvantages in carrying this feature too far, such as the increase in axial length of the runner hub and blades, with resulting increase in weight of runner and the distance of its center of gravity from the bearing, increase in length of the surrounding ring, and greater loss in friction against the ring.
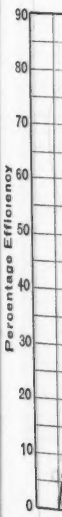
In considering the last factor, the change in form of draft-tube, no attempt at a general analysis of the effect of form of draft-tube on the performance of turbines of the propeller type will be given in this paper, as the subject has been rather exhaustively treated in other papers; it may be pertinent to point out, however, that one of the objects of recent studies on the draft-tube feature of these turbines has been the reduction in over-all dimensions without impairing the performance. To give an idea of what has been accomplished in this direction there are shown in Fig. 12 a model of the propeller type tested with a spreading draft-tube of large dimensions, in comparison with a recent design of a spreading spiral tube tested with the same model; and, in Fig. 13, the performance curves are compared.

Typical Test Results on Propeller Type Models

In Figs. 14 and 15 curves of efficiency plotted against specific speed are shown for two turbines of the propeller type of the form developed by the writer, from Holyoke tests. Each of the individual curves on these sheets represents a particular gate-opening; the envelope shows the efficiency obtainable at various specific speeds.
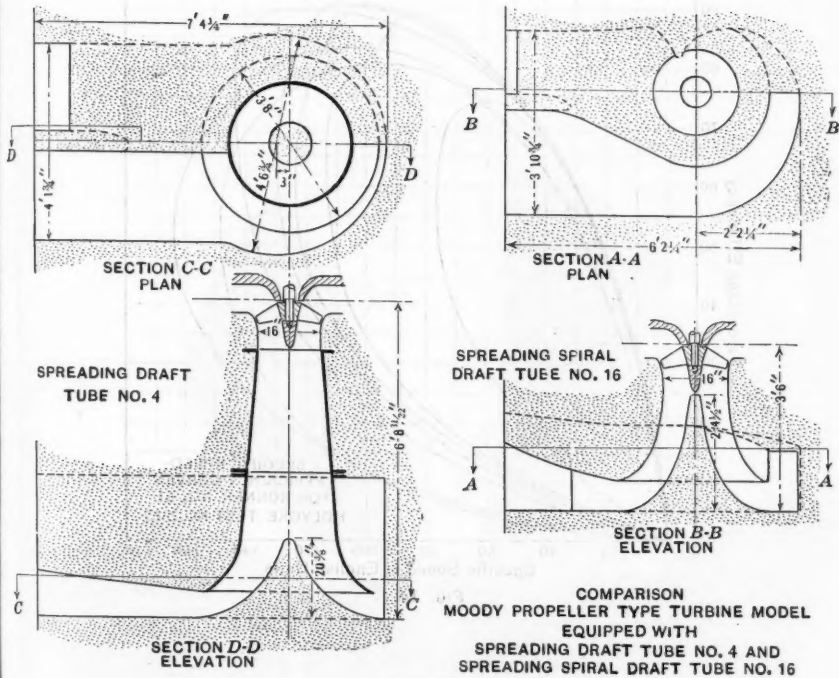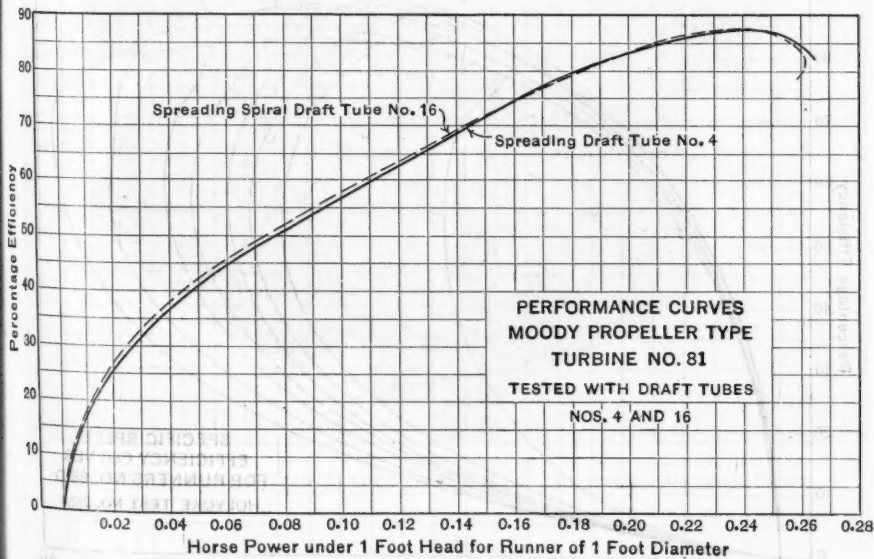
COMPARISON
MOODY PROPELLER TYPE TURBINE MODEL
EQUIPPED WITH
SPREADING DRAFT TUBE NO. 4 AND
SPREADING SPIRAL DRAFT TUBE NO. 16

FIG. 12.

PERFORMANCE CURVES
MOODY PROPELLER TYPE
TURBINE NO. 81
TESTED WITH DRAFT TUBES
NOS. 4 AND 16

Horse Power under 1 Foot Head for Runner of 1 Foot Diameter

FIG. 13.

FIG. 14.

SPECIFIC SPEED-
EFFICIENCY CURVES
FOR RUNNER NO. 81
HOLYOKE TEST NO.2862

FIG. 15.

SPECIFIC SPEED
EFFICIENCY CURVES
FOR RUNNERS NO. 68-D
HOLYOKE TEST NO. 2854
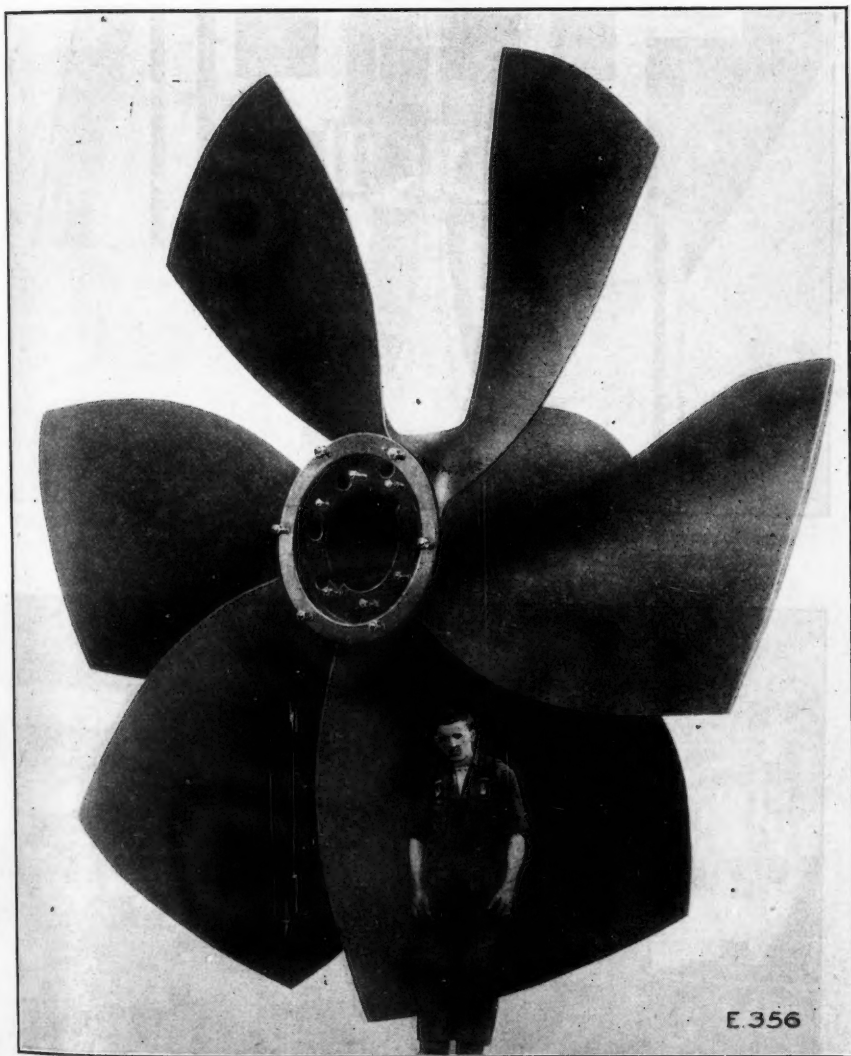
E. 356

FIG 16.—PROPELLER TYPE RUNNER (30 000 H.P.), FOR THE LA GABELLE PLANT.

Fig. 14.—Propeller Type Runner (30,000 H.P.), for the La Gabelle Plant

FIG. 17.—SECTIONAL ELEVATION OF 1 500-H.P. TURBINES FOR MOREAU MANUFACTURING
COMPANY.

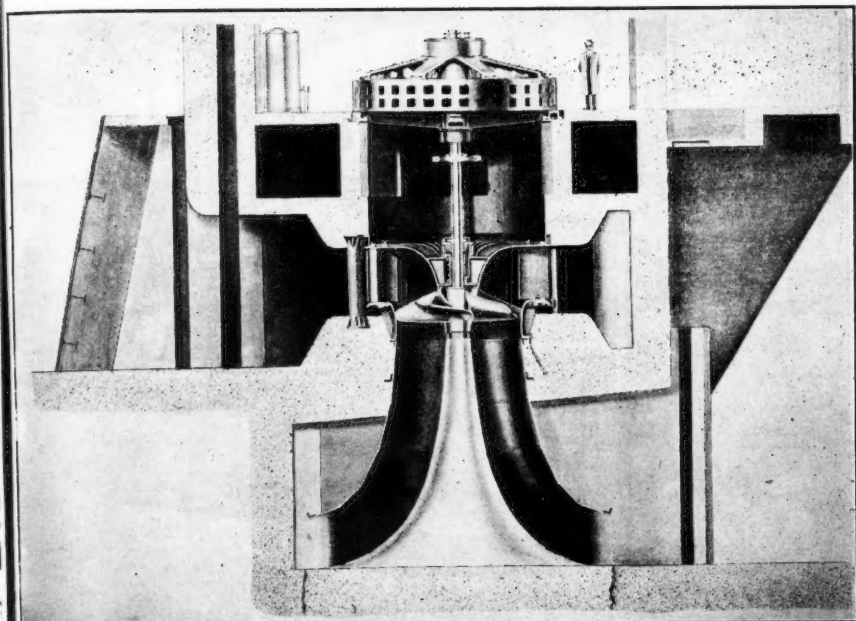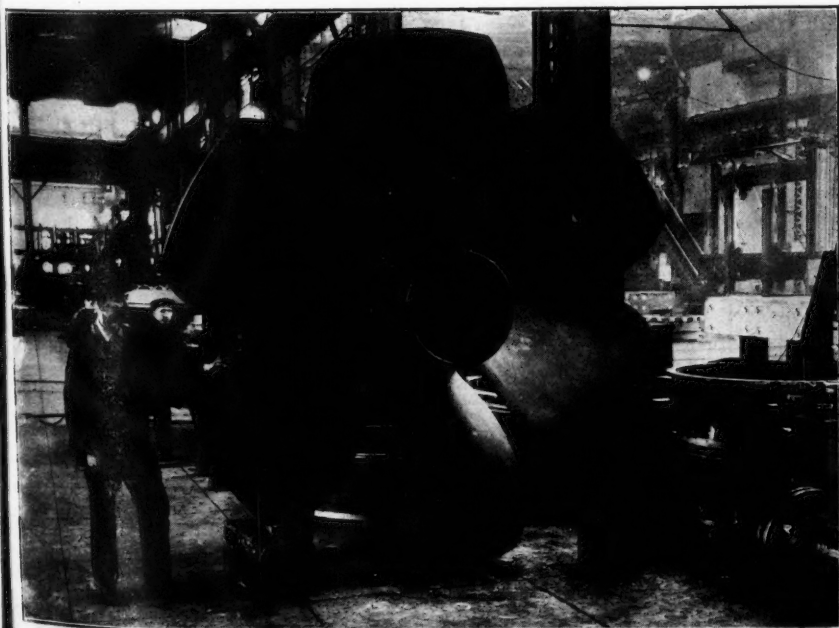FIG. 18.—PROPELLER TYPE RUNNER FOR MOREAU MANUFACTURING COMPANY.
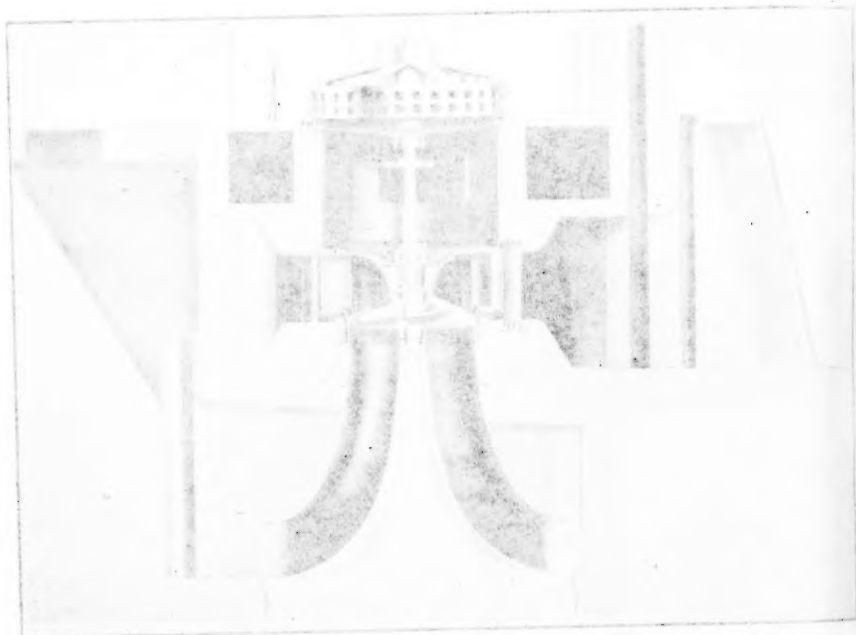
Fig. 17.—SECTIONAL ELEVATION OF PROPELLER TURBINE FOR MORRIS MANUFACTURING COMPANY.

Fig. 18.—PROPELLER TYPE RUNNER FOR MORRIS MANUFACTURING COMPANY.

FIG. 19.—INTERIOR VIEW OF POWER-HOUSE, FEEDER DAM DEVELOPMENT, MOREAU
MANUFACTURING COMPANY.

FIG. 20.—INTERIOR VIEW OF POWER-HOUSE, ANSON PLANT.

Fig. 19.—Interior View of Powerhouse, showing Main Generators, Niagara Hydro-electric Station.

Fig. 20.—Interior View of Power-House, Anson Plant.

SOME TYPICAL INSTALLATIONS

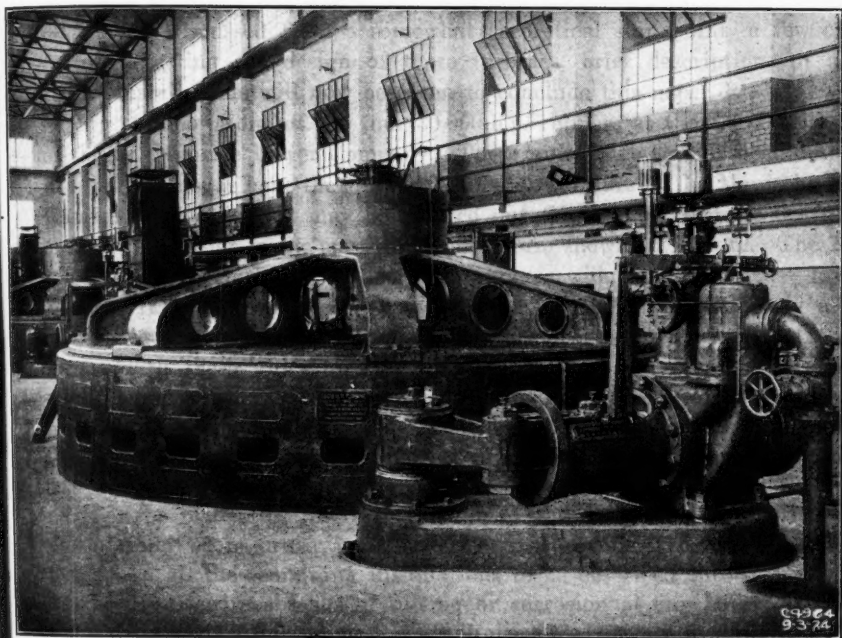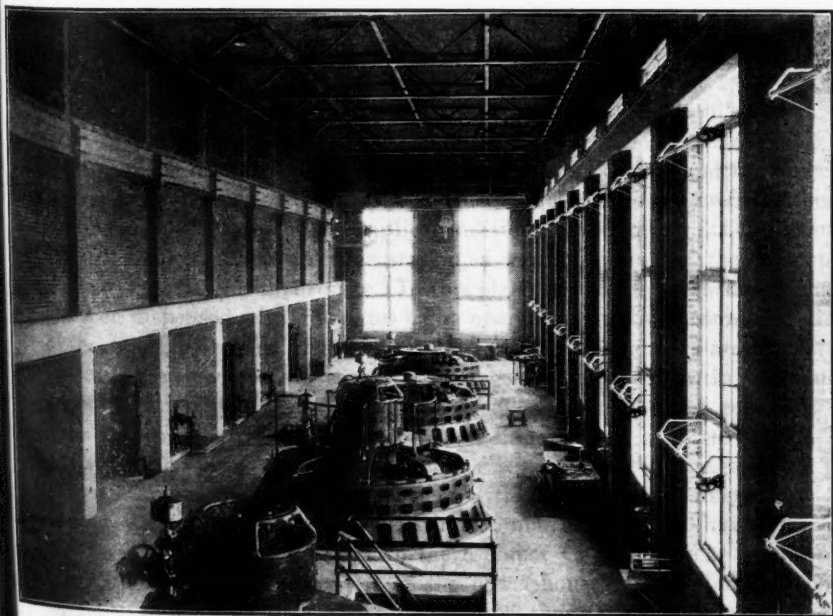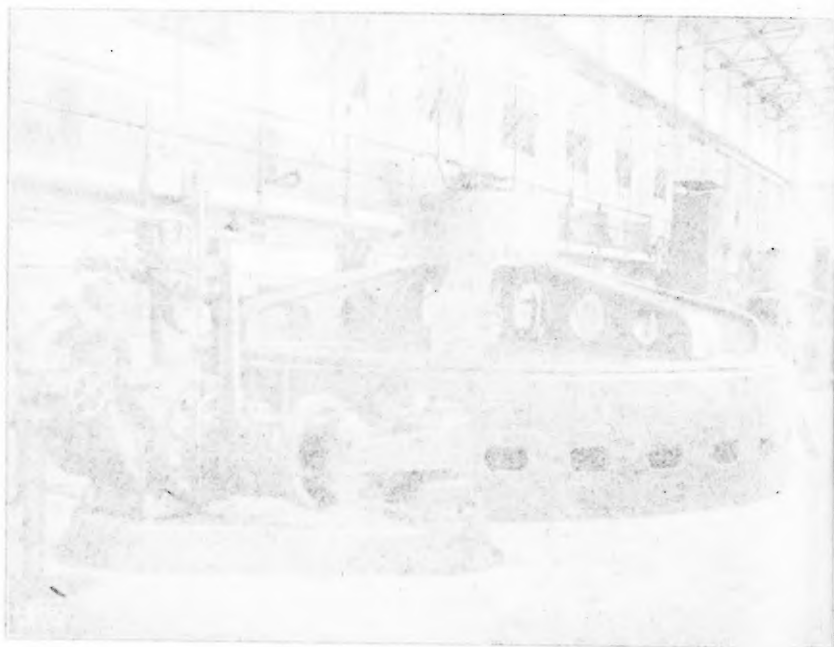Having discussed from a somewhat theoretical standpoint, a few of the factors affecting the design of these turbines, brief descriptions of a few recent installations will now be given to conclude this paper.

Fig. 16 shows one of the four 30 000-h.p. runners at the La Gabelle plant of the St. Maurice Power Company. These units have already been described in other papers, as have the similar 28 000-h.p. turbines of the Manitoba Power Company, Limited. It may be pointed out that the La Gabelle draft-tubes are closely similar to the spreading-spiral form shown in Fig. 12. The Manitoba units have now been in operation under a wide range of heads—to more than 50 ft.—for about 2 years, and the La Gabelle units for nearly a year, under heads to more than 60 ft. It may be reported as a result of the experience with these units to date that the Manitoba runners have shown a slight tendency toward corrosion on the discharge edges of the blades, due perhaps to the abnormal operating heads, but the effect is local and not serious. The La Gabelle runners are in perfect condition. In both plants the concrete just below the runner has been eroded, and this has been corrected by the addition of plate-steel liners at the top of the draft-tube, a practice which will be followed in future installations of the propeller type under the higher heads. In all the remaining installations to be described subsequently, there has been an entire absence of pitting or corrosion at any point; and in all these installations, including the Manitoba and La Gabelle units, the operation has been characterized by quietness, steadiness, and freedom from surging or vibration or other difficulties, and in every case the calculated power output has been exceeded by a reasonable margin.

Figs. 17, 18, and 19 show the 1 500-h.p. units at the Feeder Dam plant of the Moreau Manufacturing Corporation on the Hudson River, near Glens Falls, N. Y. There are five of these units with a rated capacity of 1 500 h.p. each, operating under a normal head of 15.5 ft. and a maximum head of 19 ft. The speed is 120 rev. per min., corresponding to a specific speed of 151.5 (672 metric). The design has been simplified as far as possible and the object has been to keep the cost of turbines as low as possible without sacrificing mechanical dependability or the hydraulic conditions of operation. On account of the low head available for this station, a minimum cost of the development is naturally of great importance. In this plant an outside operating mechanism has been dispensed with, the gates being operated from links connected directly to the vanes, by means of a vertical gate-shaft. The runners are of cast iron, the casing stay-vanes are separate castings set individually in the concrete, and no continuous "speed ring" is used. The draft-tube is a relatively short spreading tube with the barrel or spreading portion of plate steel. These units have been in actual operation for several months and tests of power output have shown excellent agreement with the predicted values.

Fig. 21 shows a sectional elevation of one of the 1 500-h.p. turbines for the Anson plant of the Great Northern Paper Company, near Madison, Me. Fig. 20 shows an interior view of the power-house. This plant has been in operation for about fourteen months. Fig. 22 shows the efficiency curve secured

Some Typical Installations.

Floor Line El. 102.5'

9'-7½"   24 13/32"

21" Dia.

10½"

9½"

12'-6"

11-4½"

7'-0"

C.L.Dist. El.89.0'

3'-6"

3'-6"

5¼"

7'-8" Throat Dia.

**SECTIONAL ELEVATION
1500 H.P. TURBINE
ANSON PLANT
GREAT NORTHERN PAPER CO.**
BUILT BY WM. CRAMP & SONS
SHIP & ENGINE BUILDING CO.

FIG. 21.

Fig. 21 shows a sectional elevation of one of the 1 500 h.p. turbines for the Anson plant of the *Great Northern Paper Company, near Madison, Me.* Fig. shows an interior view of the power house. This plant has been in operation for about fourteen months. Fig. 22 shows the efficiency curve secured
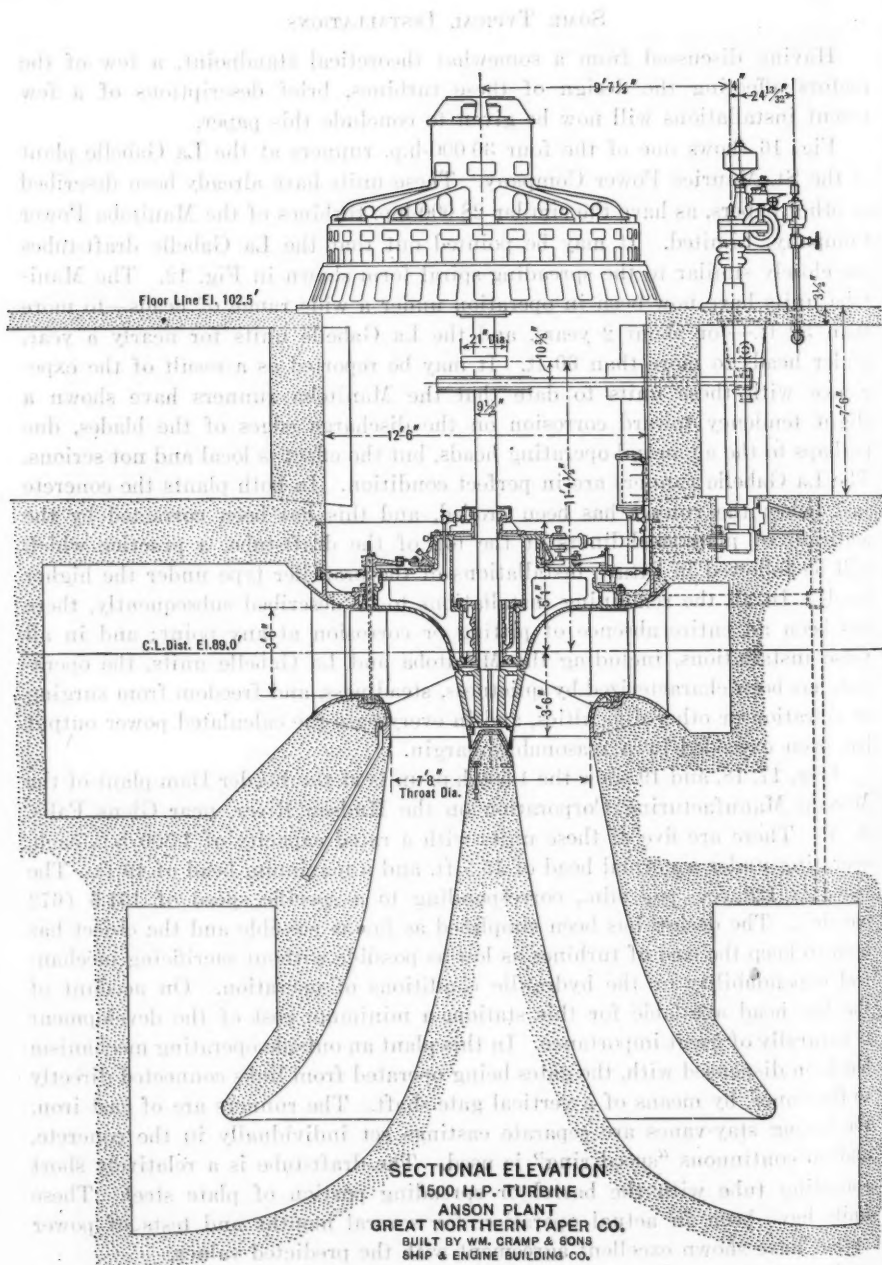
in a test of one of the full-sized units under actual operating conditions. This test was made by C. M. Allen, M. Am. Soc. C. E., using his salt-velocity method of water measurement, the water being measured in an open flume constructed in the tail-race. There are five of these units of 1 500-h.p. capacity each, operating under a normal head of 20 ft., at a speed of 150 rev. per. min. This speed corresponds to a specific speed of 137 (610 metric). These turbines have integral stay-vane rings or speed rings, outside operating mechanism actuated by a vertical gate-shaft, and the runners are of cast iron. The draft-tubes are of the spreading type.
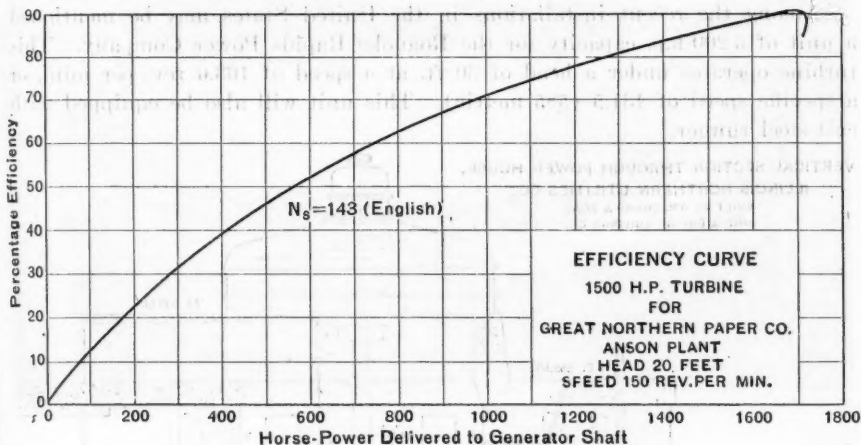

FIG. 22.

An installation very similar in design to the Feeder Dam and Anson plants is that of the Spruce Falls Company, Limited, at Kapuskasing, Ontario, Canada. This plant was built by the Dominion Engineering Works, Limited, and involves a propeller type unit of 2 500 h.p., under a 30-ft. head, operating at a speed of 180 rev. per min. This corresponds to a specific speed of 128 (571 metric). Preliminary reports of a test made by Professor Allen indicate that the efficiency will probably be in excess of 90 per cent.

Other recent installations of the same type of turbine carried out in Canada are the following:

Two units for the Howard Smith Paper Company, of 350-h.p. capacity each, under a head of 8 ft. These have a speed of 99 rev. per min., corresponding to a specific speed of 134 (597.5 metric).

One unit for the Dryden Paper Company of 1 400-h.p. capacity, under a head of 29 ft., at a speed of 225 rev. per min., corresponding to a specific speed of 125 (556 metric).

These installations were built by the Dominion Engineering Works, Limited, of Montreal, Que., Canada, which also has under construction two turbines of 6 000-h.p. unit capacity for a head of 30 ft. normal, and 34 ft. maximum for the Southern Canada Power Company. The draft-tubes are of the spreading type formed in concrete and will be provided with separate cast stay-vanes at the end of the spreading portion to carry the weight of the

superposed structure. Similar stay-vanes were used in the Manitoba and
La Gabelle installations and in a large number of Francis turbine installations.
These vanes have been found to be a valuable structural feature reducing the
reinforcement in the power-house substructure which would otherwise have
to carry the loads by beam action, and also proving useful during construc-
tion as they permit the earlier removal of forms. These turbines will have
removable liner rings surrounding the runners; the runners will be of cast
steel. The operating mechanism will be of the outside type. These units
will operate at 138.5 rev. per min., or a specific speed of 153 (680 metric).

Among the recent installations in the United States may be mentioned
a unit of 3 200-h.p. capacity for the Roanoke Rapids Power Company. This
turbine operates under a head of 30 ft. at a speed of 163.6 rev. per min., or
a specific speed of 131.5 (585 metric). This unit will also be equipped with
cast-steel runner.

VERTICAL SECTION THROUGH POWER HOUSE,
ILLINOIS NORTHERN UTILITIES CO.
BUILT BY WM. CRAMP & SONS
SHIP & ENGINE BUILDING CO.
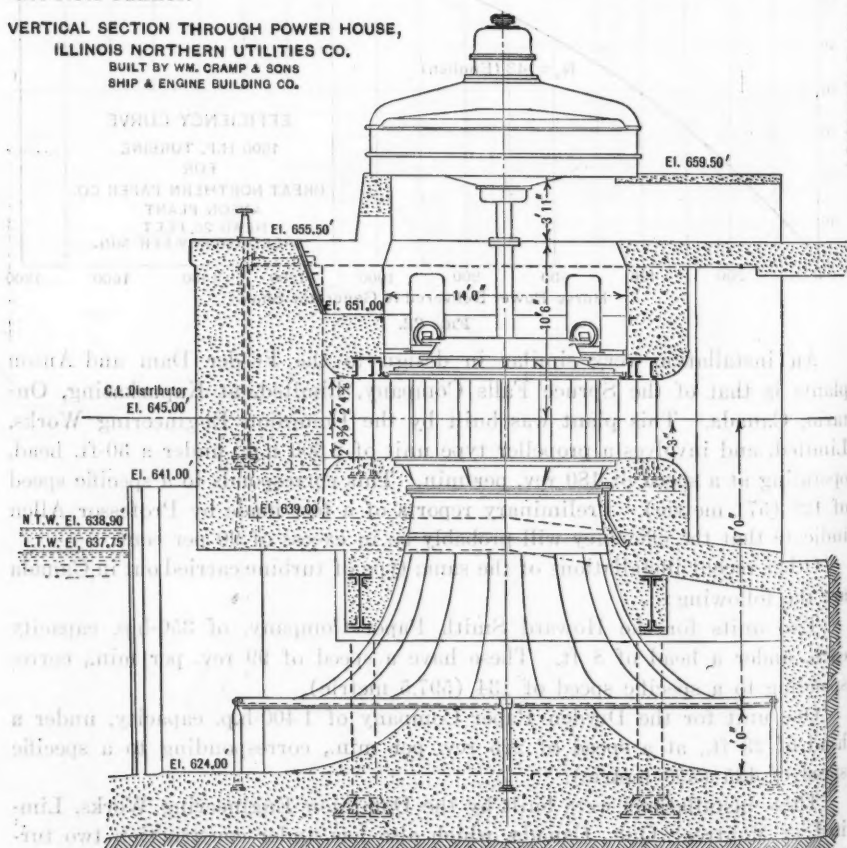
FIG. 23.

An installation of particular interest, the construction of which has recently
been completed, is that of five units for the Illinois Northern Utilities Com-
pany, shown in section in Fig. 23. These turbines will have a unit capacity

Papers.

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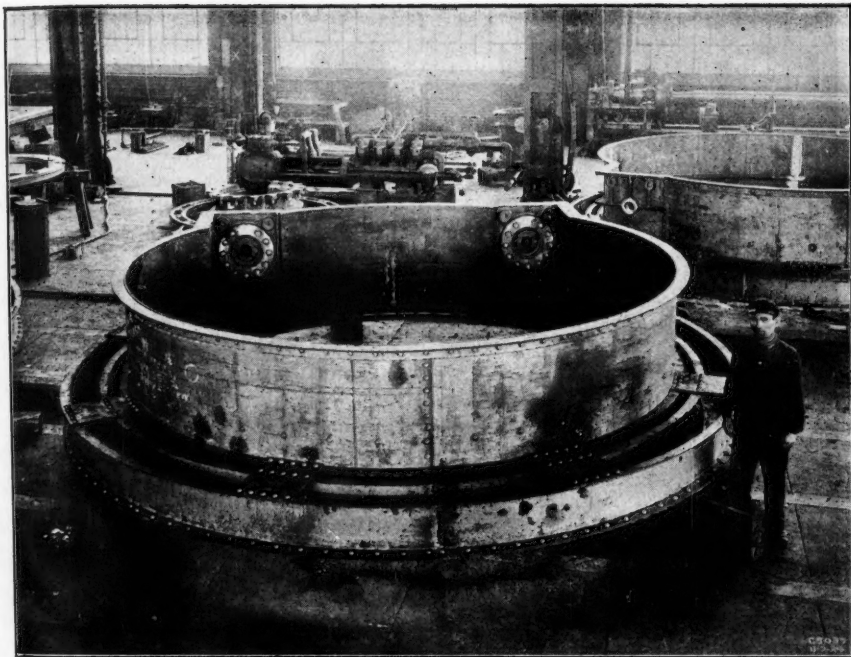
FIG. 24.—FOUNDATION RING, ILLINOIS NORTHERN UTILITIES COMPANY.

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apacity
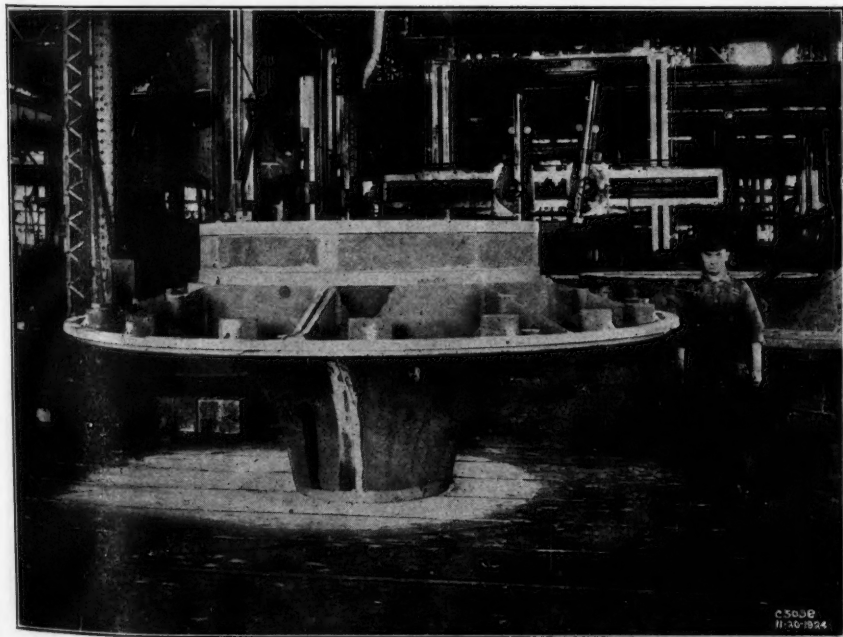
FIG. 25.—TURBINE HEAD COVER, ILLINOIS NORTHERN UTILITIES COMPANY.
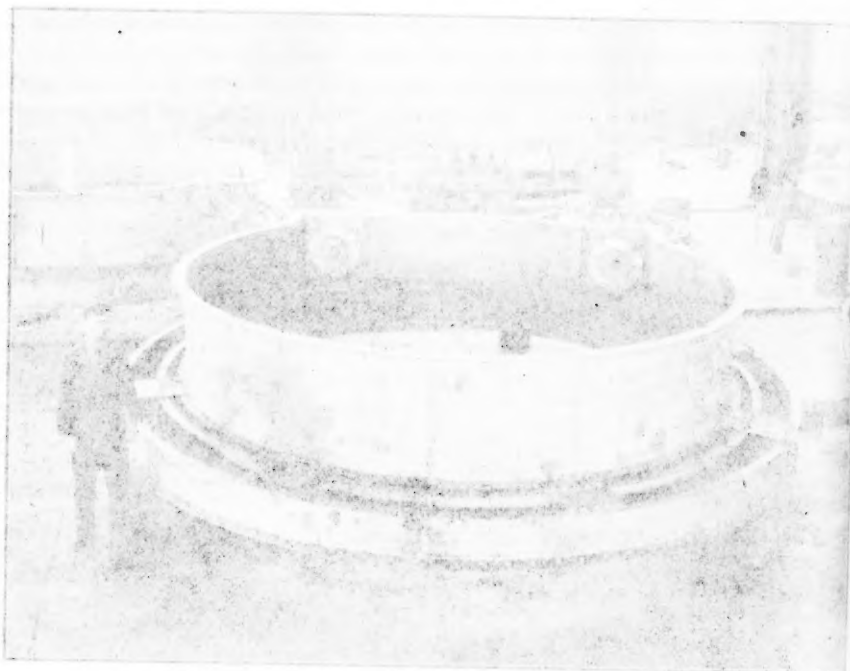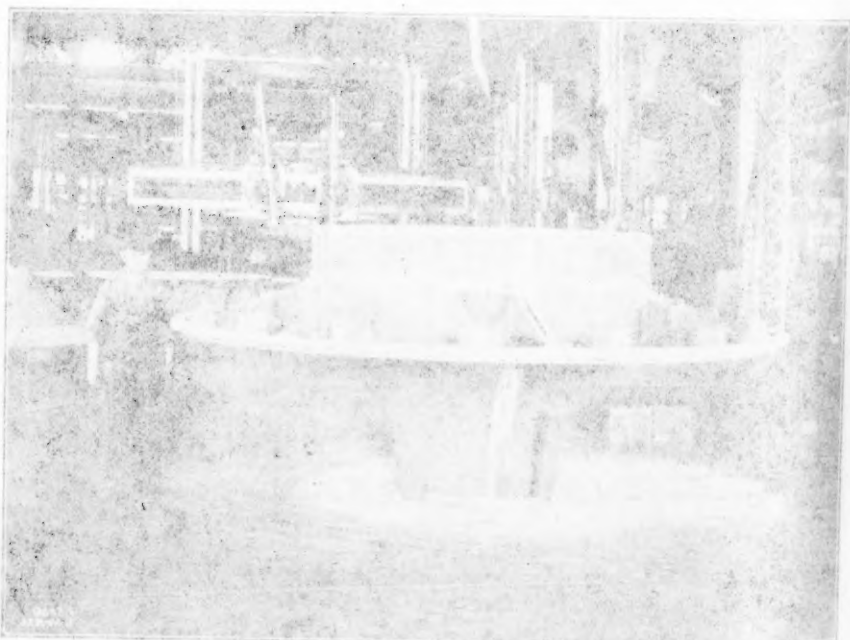
FIG. 26.—GUIDE-VANES, ILLINOIS NORTHERN UTILITIES COMPANY.
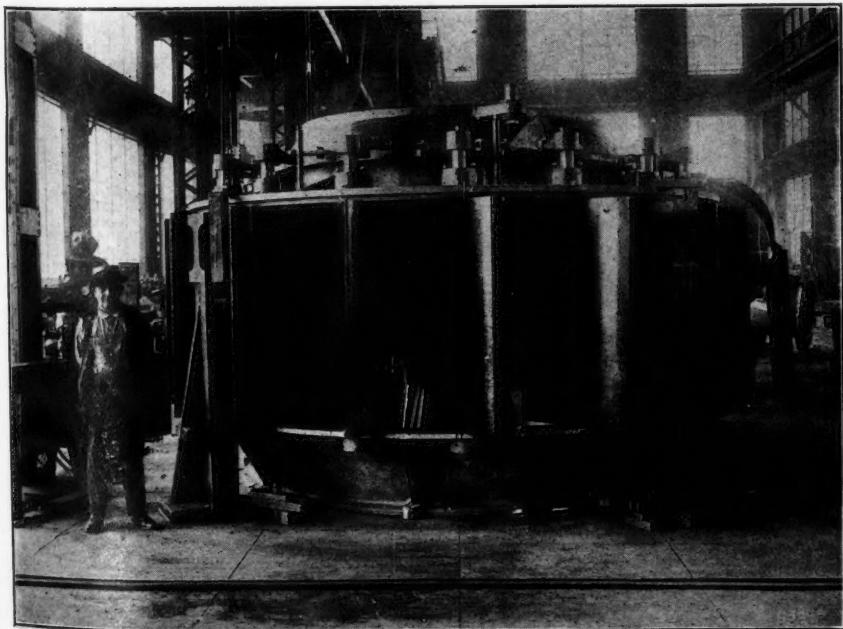
FIG. 27.—PARTIAL ASSEMBLY OF 800-H.P. TURBINE FOR ILLINOIS NORTHERN UTILITIES COMPANY.

FIG. 20.—CROSS-VALVE, ILLINOIS NORTHERN UTILITIES COMPANY.
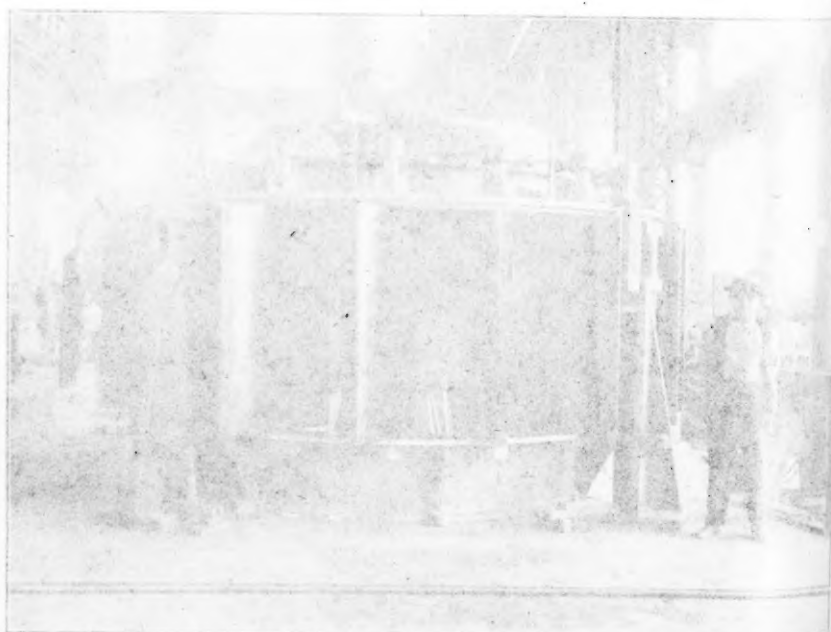
FIG. —PARTIAL ASSEMBLY OF 500-KW. TURBINE FOR ILLINOIS NORTHERN UTILITIES COMPANY.

of 800 h.p., under a head of 8 ft., operating at a speed of 80 rev. per min. This plant is of unusual interest because of the low head which has been utilized. The specific speed is 168 (748 metric). Every effort was made to keep the cost of installation to a minimum. Separate stay-vanes have been used, bolted at the top to the foundation ring. This foundation ring is built up of plates and structural shapes and is continuous with a plate-steel pit liner carrying the operating cylinders as shown in Fig. 24. Fig. 25 shows the turbine head cover. Fig. 26 shows the guide-vanes, which are built up of plate steel with cast-steel stems. Fig. 27 shows the turbine partly assembled in the shop. As in all the other units described in this paper, the draft-tubes are of the spreading type and in this installation are constructed of plate steel. In all the later installations, the draft-tubes have been of the continuous annular type with central cores continuous in contour with the runner hub.

CONCLUSION

It may be stated that although as a result of experience with this new form of turbine, certain minor defects have developed which have required for their correction some modifications in design, the many major difficulties which had been predicted, such as serious corrosion, vibration, instability, imperfect regulation, failure to develop the power corresponding to the head, or refusal to run under low head, have not occurred.

HIGH-SPEED SUCTION TURBINES

By Forrest Nagler,* M. Am. Soc. C. E.

Synopsis

This paper is intended to outline in a general way the experience to date with the high-speed, axial-flow type of hydraulic turbine runner for low heads. A brief comment as to the origin of this type is given, together with a statement as to the number of installations, total horse-power involved, etc. Comment is given as to the operating features that have been observed, the heads to which this type has been applied, and the present commercial range of its application.

Definition

The high-speed axial-flow type of hydraulic turbine runner, called variously in the United States the "suction runner" or "propeller runner", is generally representative of a type wherein the flow within the runner itself is in a direction mainly parallel to or "cylindrical" about the axis of rotation. (See Fig. 28.)

This is in contrast to the usual types where the flow is concentric about, but transverse to, the axis of rotation as in the Francis types (Fig. 29 left), or at right angles to the axis, but not concentric about it, as in the "tangential impulse", or "Pelton" type (Fig. 29 extreme right). The particular design for which the writer has been responsible is further characterized by a small number of blades (usually and preferably four), as contrasted with the fifteen to thirty prevalent in present conventional forms (see Fig. 29), and by the absence of a rim around the tips of the blades. In addition, there is a reduction in the width of the blade so that for the higher speed conditions there is no overlap, this being essential to highest efficiency when high characteristic speeds are reached. It is with the type thus defined, that this paper has to deal.

Historical

The place which this type of runner holds historically, particularly with reference to all other types, is shown in Fig. 31. In plotting this diagram, every obtainable performance figure has been used for the earlier forms. The writer believes this diagram to be still correct, although a paper† presented before the Society would indicate otherwise.

In that most excellent paper Messrs. Safford and Hamilton made the following statement,‡ under the heading, "The Propeller Type of Runner."

"* * * wheels of almost exactly the same type were used many years ago, and that their high speed was appreciated. * * * Not only did Truax

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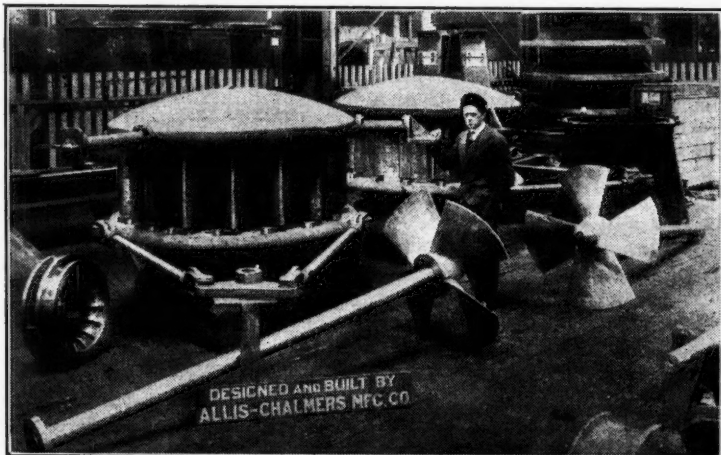
FIG. 28.—TYPICAL HIGH-SPEED SUCTION RUNNER (1916). COMPARE WITH
FRANCIS WHEEL OF ABOUT ONE-EIGHTH CHARACTERISTIC SPEED.
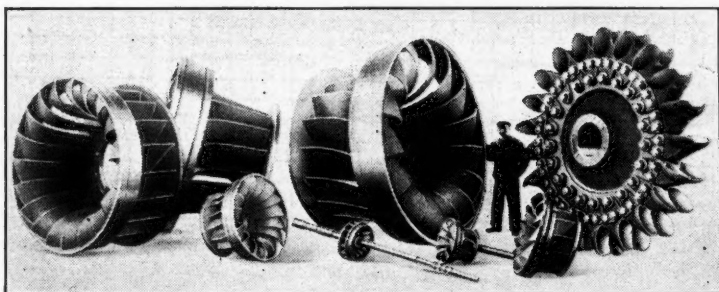
FIG. 29.—TYPES OF WATER WHEELS FOR HEADS OF 2 000 FT. TO 8 FT.
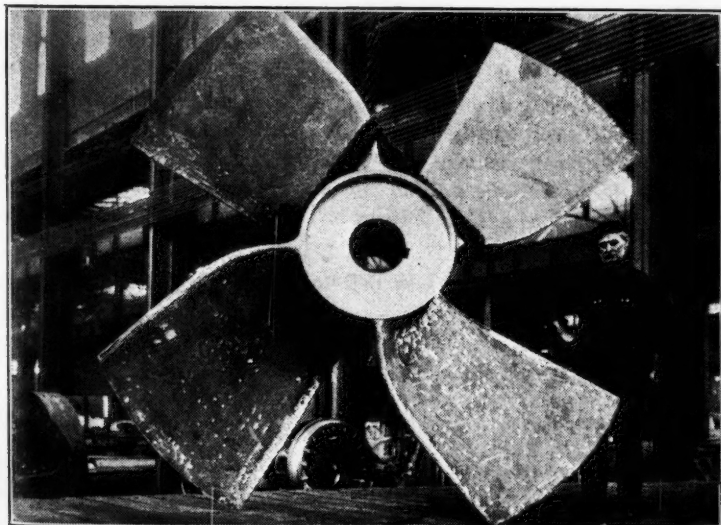
FIG. 30.—NAGLER 116-IN. HIGH-SPEED RUNNER, CAST STEEL IN ONE PIECE.

appreciate what he had, but he also knew how he got it, which was unusual at this time. * * * Assuming that the measurement of the revolutions per minute and of the water was correct, and, furthermore, assuming an efficiency of 60%, which seems fair and reasonable in the light of the performance of similar wheels of the same period, a specific speed of 125 is obtained. If the efficiency has been only 40%, the specific speed would still be more than 100."

Lest this statement from so well known an authority give rise to a mis-apprehension as to the number of years of experience behind this type of run-ner, the following comments are thought to be in order. They are offered for the purpose of avoiding too rapid an application of this type to conditions for which experience has not yet proven it economical.
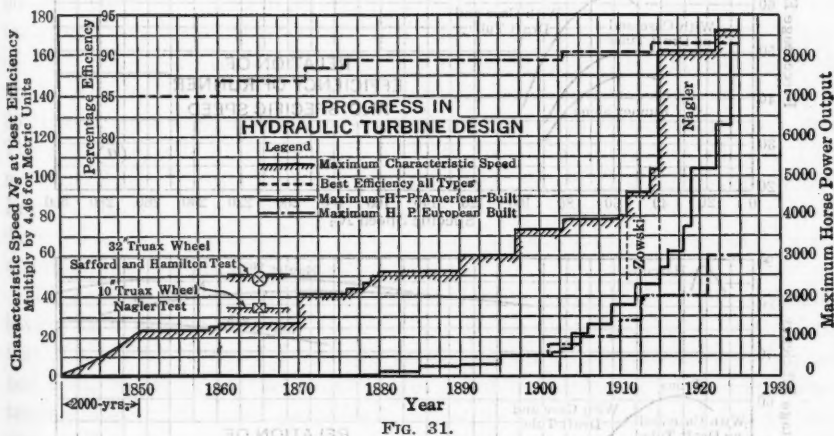
FIG. 31.

The writer possessed 10-in. wheels of the Truax and Austin (similar to the Truax) types; he made tests of these wheels and gave their results in detail in discussion* of the paper by Messrs. Safford and Hamilton. Specific speeds of 35 and 23 were obtained, using settings duplicating as nearly as possible those shown in the original Truax and Austin illustrations. Messrs. Safford and Hamilton took most vigorous exception to these tests; they secured a 32-in. Truax wheel and tested it, without finding specific speeds in excess of 60, even with a draft-tube added. The results are shown in Table 2.

TABLE 2.—COMPARISON OF TRUAX AND NAGLER WHEELS.

Wheel test.	Efficiency, percentage.	Specific speed (English units).
Original Truax advertisement....................................	60 (assumed)	125 (calculated)
Original Truax advertisement....................................	40 (assumed)	100 (calculated)
10-in. Truax, Nagler test.......................................	38 (by test)	35 (by test)
32-in. Truax, Safford and Hamilton test.........................	50 (by test)	51 (by test)
32-in. Truax plus draft-tube, etc., Safford and Hamilton test........	61 (by test)	60 (by test)
Nagler's original 34-in. runner, 1915.............................	87 (by test)	150 (by test)

* Transactions, Am. Soc. C. E., Vol. LXXXV (1922), p. 1310.

Replotting this comparative data on a specific speed (N_s) basis, gives the curves shown on Fig. 32(a). Another basis is given in Fig. 32(b), in which the use of unit horse-power exaggerates the comparison even as the speed basis suppresses it. The N_s basis is the fairer contrast.
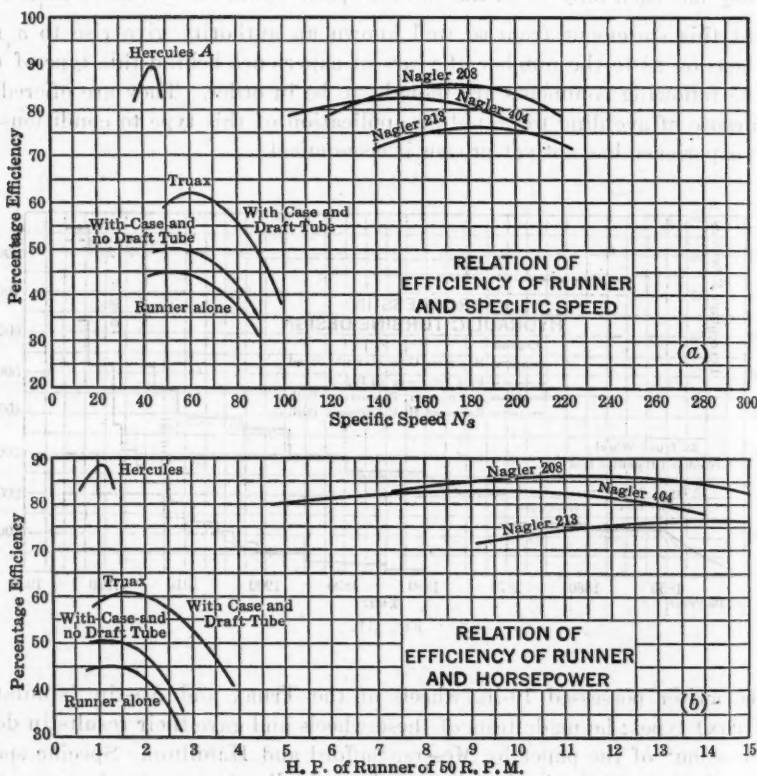
FIG. 32.

This comparison, particularly as it includes tests with additions in the form of guide-cases and draft-tubes built in the light of present knowledge, seems to the writer to show conclusively that there is some decided difference between a type of wheel which tests only to 60% efficiency and a specific speed of 61, and the modern suction type as here presented which reaches efficiencies of 90%, and specific speeds from 150 to 200, or about triple that of the earlier type.

The summary diagram, Fig. 60*, of Messrs. Safford and Hamilton, is entirely misleading. In the first place, the curve labeled "Truax" should be labeled "Truax 1876 plus Safford and Hamilton's Guide-Case and Draft-Tube, 1922". Even neglecting this point, the Truax wheel with a specific speed of 60 at the best efficiency point, represents more closely the class including the Hercules, which ranges in specific speed from 40 to 70, than it does the writer's

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1349.

wheel with its specific speed of 150 to 200. The Truax curve projected for over-speed extends up to a specific speed of less than 100, but it should be pointed out that the wheel which the writer contends is different, extends similarly up to 250.

PERFORMANCE

The writer's experimental work on this type of runner as applied to hydraulic turbines was started in 1913. The initial plant (Fig. 28) was built in 1916, the initial Holyoke test (No. 2 521) was made in January, 1917, and the first publicity was given to this type in 1919.* At that time a total of 17 units had been built, 9 of which had been tested. To date more than 100 of these units have been built, 90 of which have a total combined capacity of 90 680 h.p. The operation of every plant has been watched most carefully and tests have been made wherever possible. The sizes have ranged from a diameter of 30 in. up to one of 156 in. The horse-power per unit has ranged from 100 up to slightly less than 5 000. The heads have varied from 22 in. up to 35 ft. and, on test plants, up to 65 ft., these high-head tests dating from 1920. The specific speeds have varied from 90 to 250, English system, (400 to 1 100 metric). Holyoke tests have not been made as frequently as is customary with Francis units. The highest Holyoke efficiency reached was on a runner designed in 1920 and tested in 1923, showing a maximum efficiency of 89.2 and a specific speed at best efficiency of 175.

In general, plants have been operated with remarkably little difficulty, having an over-all continuity or reliability factor not noticeably different from that of the Francis type of unit, as far as mechanical features are concerned, and being superior to the Francis type from a standpoint of freedom from troubles due to ice, trash, and débris, either within the turbine or in the trash racks. There has been no replacement of any runner due to mechanical difficulties, nor a single instance of pitting, although practically every installation of any considerable head has been carefully watched for this trouble.

A typical performance curve is shown in Fig. 33, wherein it will be noted that a maximum hydraulic turbine efficiency of 91% is reached in the field. The runner of this particular unit is shown in Fig. 30.

CHARACTERISTICS AND LIMITATIONS

Practically all installations have followed the general practice conventional for the Francis type of runners, as concerns the guide-case, flume, or spiral casing. On the discharge side, practically every installation has made use of the White hydraucone regainer,† an efficient diffuser being essential by reason of the higher discharge velocity from this type of runner and the consequent necessity for as efficient a regain as possible.

One of the principal characteristics of this type of runner, is its lower efficiency at partial loads, as contrasted with those of lower speed, such as the Francis type. For example, the efficiency at half load of a high-speed runner having a characteristic speed of 175 is about 66 per cent. The corre-

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* "A New Type of Hydraulic Turbine Runner for Low Head Installations", by Forrest Nagler, M. Am. Soc. C. E., *Transactions*, Am. Soc. Mech. Engrs., Vol. 41 (1919), p. 829.
† *Transactions*, Am. Soc. Mech. Engrs., Vol. 43 (1921), p. 255.

PERFORMANCE CURVES OF
**ALLIS CHALMERS
HIGH SPEED RUNNER**
Runner No.54, 116 in. diam.
2500 B.H.P. at 138.5 rev.
per min. and 21 ft. head
Tested at heads of 20.5 ft.
to 21.1 ft.

Legend:
Power House Results
Approx. from Holy. Test
Guaranteed Efficiency
o o o Test Points

Efficiency

Discharge

Horsepower

Guaranteed Output

Percentage Efficiency

Discharge Cu. Ft. per Sec. 1 ft. Head
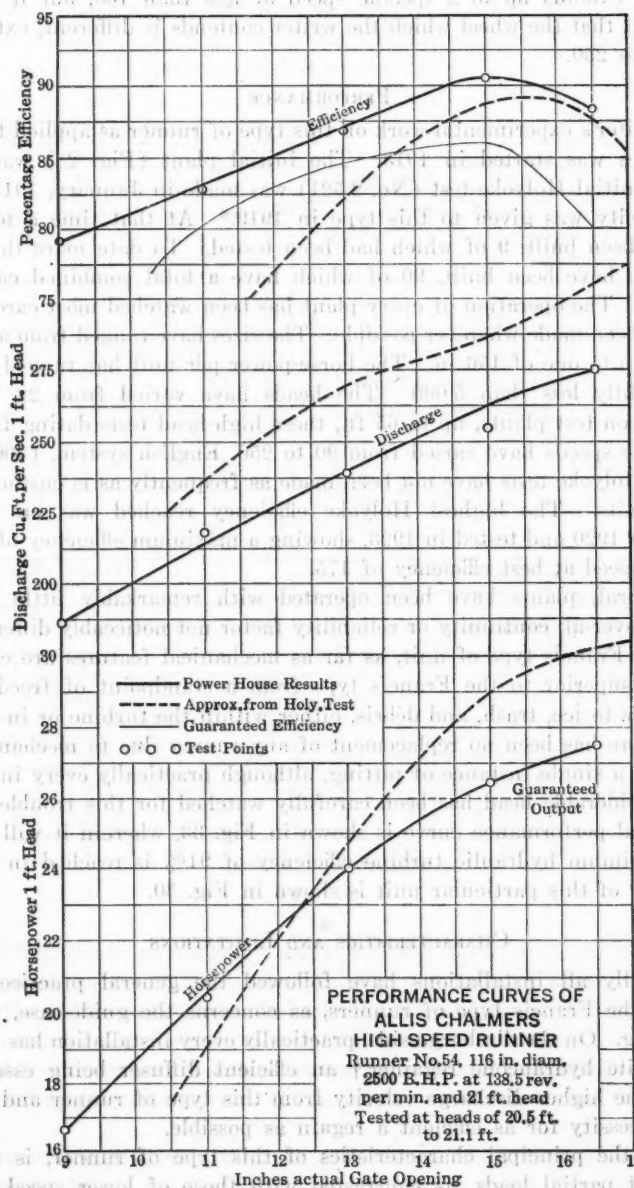
Horsepower 1 ft. Head

Inches actual Gate Opening

Fig. 33.
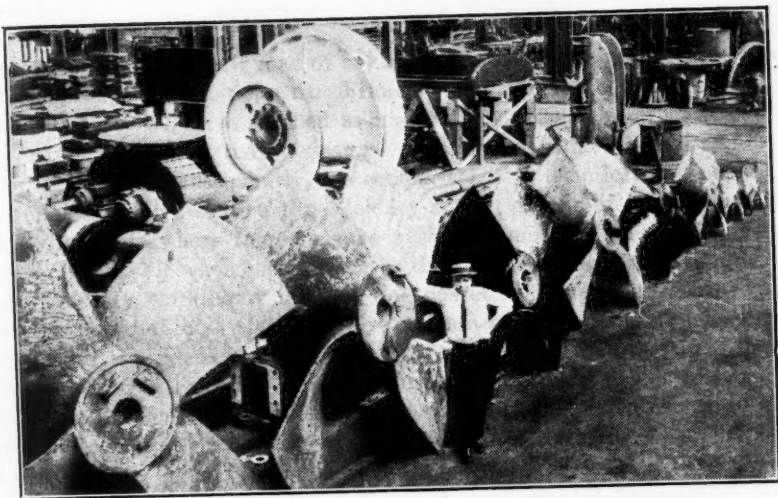
Fig. 34.—Examples of Modern High-Speed Suction Runners.
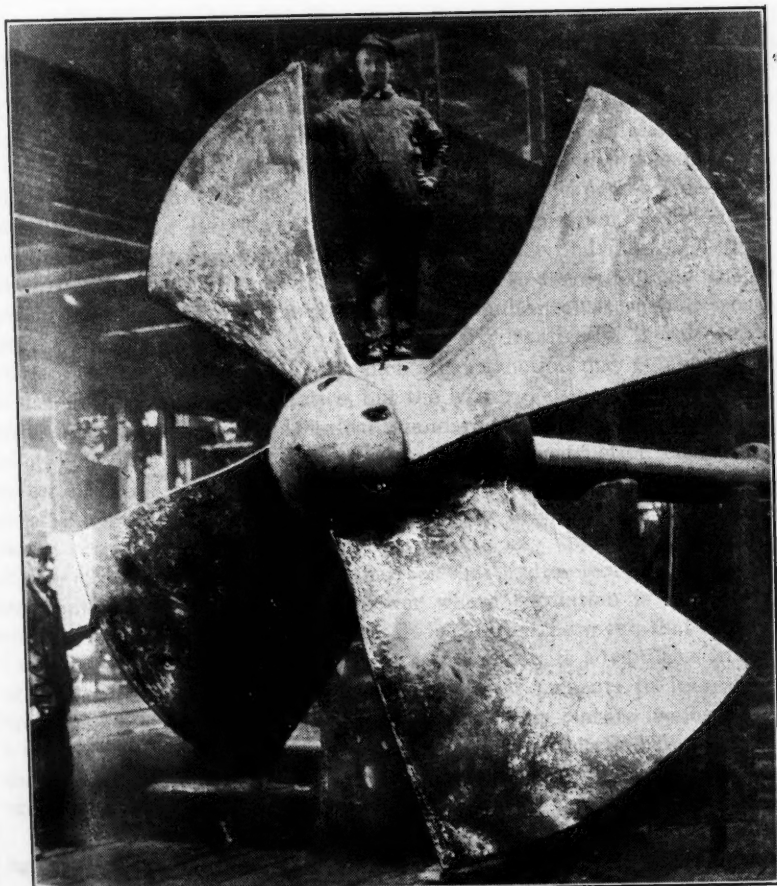
Fig. 35.—Largest High-Speed Suction Runner Ever Installed for Low Heads.

sponding efficiency at half load of a Francis runner having a specific speed of 100 is approximately 75 per cent. Francis runners of low capacity, for specific speeds of 40 or 50, have reached as high as 90% efficiency at half load, but these runners have not the characteristics that will permit of their application to low heads and, consequently, the comparison is a technical and not a practical one. In actual service, the efficiency at part gate is little or no handicap, except in plants where there may be a small number of units. It is not a considerable factor when there are more than two units in the station, because under ordinary working conditions, the operation may be maintained at or near the point of best efficiency, by intermittently shifting the load or by drawing on the pondage. This possibility combined with the fact that the over-all operating efficiency of the high-speed unit under low-head conditions (including the generator) is usually superior to that of a Francis unit, makes this type economical regardless of the saving in initial cost. Figs. 30, 34 and 35 show runners typical of both horizontal and vertical installations. The contrast in Fig. 34 between high-speed suction runners and, in the background, the Francis reaction runner which they have virtually displaced for low heads, is clear. The runner shown in Fig. 35, built for Henry Ford and Son, Incorporated, Green Island development, is 156 in. in diameter, 17 000 lb. in weight, and cast in four sections. It operates at 80 rev. per min. under a head of 13 ft.; its discharge capacity is about 85% that of the Keokuk runners, which weigh about 179 000 lb. each and measure more than 17 ft. in outside diameter.

One of the features encountered earliest in designing and experimenting with this type of runner is the possibility of varying its capacity widely without materially affecting its efficiency. It is simply a question of flattening the blade angles. European practice has apparently taken advantage of this fact and carried the application of this principle even so far as to make the blades movable under control of the governor, either simultaneously with or separately from the similar control of the guide-vane. Hydraulically, there is no question as to the results; mechanically, such a construction may come well within the standards of European practice, but the writer does not believe that it will ever become popular under American standards of simplicity, of ruggedness, and of high costs of labor. The need for such a construction does not enter into consideration when there are many units in the station. Governors are becoming less and less a factor, and the writer believes they will ultimately cease to enter into hydraulic turbine design to any appreciable extent, their ultimate elimination being due to the fact that hydro-electric systems will be largely interconnected with steam power where regulation is inherently less harmful to structures and to economy. It is believed, however, that this feature of variable capacity will be utilized, not so much for its advantages in increasing part-load efficiency, as for the possibilities it presents in increasing the capacity under conditions of flood or low head. This feature has been utilized in numerous installations already built, construction details being used corresponding to the adjustment of pitch on screw-pumps, which has long been the regular practice in the United States.

It has been a fairly well defined policy of the Allis-Chalmers Manufacturing Company not to apply the suction type of runner to heads materially in

excess of 30 ft., this being an arbitrary limit, although it is supported by experimental data as well as by theoretical considerations. By departing from the pure suction type of runner with its minimum blade surface and by using blades of larger projected area, the limiting head can undoubtedly be increased, but it is the writer's belief and experience that the considerable increase of projected area, either by reason of wider blades or a larger number of blades, is accompanied by a very decided reduction of efficiency. It will undoubtedly be found that for certain installations, a definite gain in some other direction may counterbalance this particular detrimental feature and probably as experience grows, this general type will be applied to higher and higher heads and to larger and larger capacities. At present, it is not believed that it is commercially advisable or sound from an engineering standpoint to apply it to heads or capacities far outside the range of proven practice. A consistent adherence to this policy has enabled the achievement of an operating record wherein there has been no single shut-down of a unit of as much as 24-hours duration, due to conditions traceable to this type of runner.

Governor regulation has not been as perfect as that obtained with corresponding types of Francis units, but this is principally attributable to the fact that, with the higher speeds in question, the factor, $W R^2$, of the generator has been relatively lower. The regulation experienced has not differed from that obtained with Francis units by an amount unexplainable by this reduction of regulating constant.

The high-speed suction type of runner, due to its higher and sometimes critical velocities, is not conducive to as quiet a running machine as the Francis runner, perhaps in exactly the same way that the flow conditions in a Francis runner are not conducive to as smooth operation as is possible with an overshot wheel. Nothing, however, of a nature that would make it uncommercial has been experienced with this type when it is used under conditions to which it is inherently adapted, that is, relatively low head and large capacity.

HIGH SPECIFIC SPEED HYDRAULIC TURBINES IN THEIR BEARING ON THE PROPORTIONING OF THE NUMBER OF UNITS IN LOW-HEAD HYDRO-ELECTRIC PLANTS

George A. Jessop,* Assoc. M. Am. Soc. C. E.

The writer will indicate briefly the comparative efficiencies which can be obtained by using medium-speed wheels (of which the specific speed, N_s, is equal to about 75) and high-speed wheels (N_s equal to about 175). The efficiencies shown throughout this paper are based on model runners of the size usually tested at the Holyoke flume, that is, about 39 sec-ft. discharge at 1-ft. head. Under good conditions, it is possible to obtain better results with larger wheels in place. The data and information contained herein are obtained from tests made in the Holyoke flume and in the specially designed and constructed testing flume of the S. Morgan Smith Company.

The medium-speed runner used in these tests is of the Francis type and of normal design. The high-speed runner is a design first developed and used by The Theodore Bell Company, of Switzerland, and now being further developed and improved jointly by the Engineering Staffs of The Bell Company and the S. Morgan Smith Company. It is distinctive in that it has fewer blades than the usual high-speed wheel, and the total blade area is exceptionally large. The hub is also large, thus reducing the usual overhang of the buckets and providing a large cross-sectional area where each bucket joins the hub. An inspection of the curves will show that this wheel has its peak efficiency at a load of about 90% comparing very favorably with the slower speed runner in this respect. This is a desirable feature, because the turbine can be operated at its most economical point with a liberal allowance of power for regulating purposes, and, furthermore, this design assists in obtaining high part-load efficiency.

For very low heads (30 to 40 ft. and less), the comparison given herewith is possibly somewhat less favorable to the high-speed wheel than it should be, because usually for these heads, if a Francis runner is used, it will be of a specific speed of about 100 to 110, and will have a slightly lower peak efficiency and considerably lower efficiency at one-half to three-quarter load, than a wheel with N_s equal to about 75. Furthermore, this being a comparison of turbines only, the increased efficiency of the high-speed generator is not taken into account.

In this paper it will be impossible to present a full and complete discussion of this question, but a comparison will be given of the results that can be obtained with plants containing from two to eight units of the medium and high-speed types, together with some averages that it is hoped will be interesting and helpful.

Fig. 36 shows the comparison between medium and high-speed wheels in a 2-unit plant. Up to 50% load, of course, only one unit is in operation.

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* Hydr. Engr., S. Morgan Smith Co., York, Pa.

FIG. 36.

Labels in figure: Efficiency Turbine with $N_S=75$; 2-Units Operated; Efficiency Turbine with $N_S=175$; 1-Unit Operated; Turbine with $N_S=75$; Turbine with $N_S=175$; Average Efficiency Plant operating from 100% Load to any other Load between 0 and 50% Load varies at a constant Rate; EFFICIENCY 2-UNIT STATION COMPARISON OF MEDIUM AND HIGH SPEED TURBINES; Percentage Average Efficiency; Percentage Efficiency; Percentage of Fall Load

FIG. 37.

Labels in figure: Efficiency Turbine with $N_S=75$; 3-Units Operated; 4-Units Operated; 2-Units Operated; Efficiency Turbine with $N_S=175$; 1-Unit Operated; Turbine with $N_S=75$; Turbine with $N_S=175$; Average Efficiency Plant operating from 100% Load to any other Load between 0 and 50% Load varies at a constant Rate; EFFICIENCY 4-UNIT STATION COMPARISON OF MEDIUM AND HIGH SPEED TURBINES; Percentage Average Efficiency; Percentage Efficiency; Percentage of Fall Load
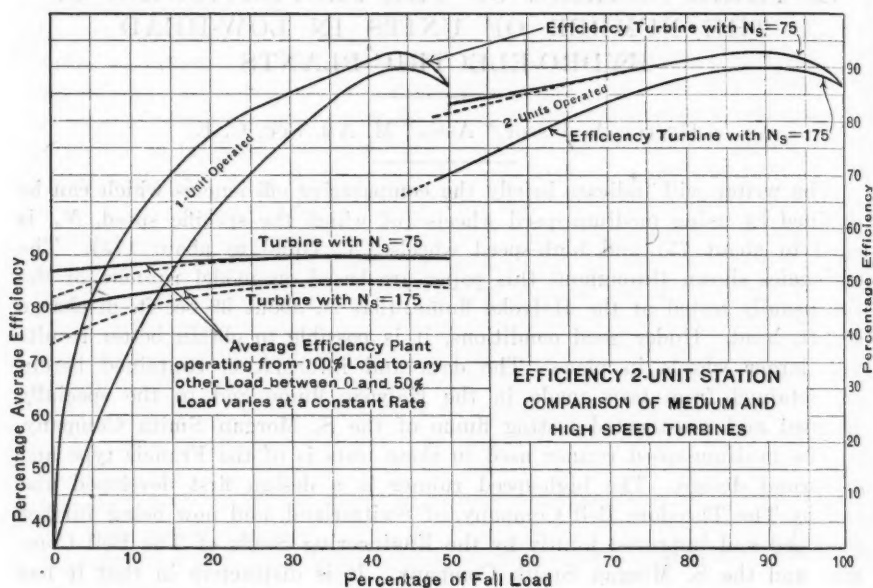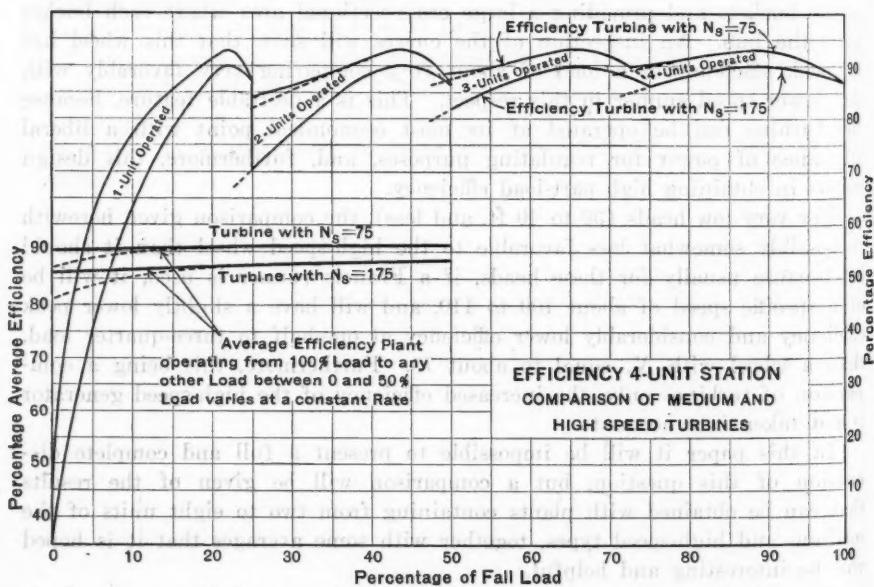
Above 50% load, the curve is the resultant or combined efficiency of the two units. For the medium-speed wheels, when operating between 50 and 75% load, a higher efficiency can be obtained by running one wheel at maximum efficiency and supplying the remainder of the demanded power by running the second unit at the required gate-opening, than by dividing the load equally between the two units. The efficiency to be gained by the unequal distribution of load is shown by the full and dotted lines (lower left, Fig. 36). The high-speed wheels show maximum efficiency when the load is equally divided. In order to determine whether it is economical to divide the load equally or unequally between two or more units, it is necessary to calculate each plant separately, using the curves which apply. On the left of Fig. 36 curves showing the average weighted efficiency have been plotted, with the station operating from 100% load to any other load between 0 and 50%, based on the load varying at a constant rate.

The weighted efficiency for a given range of load is simply the total output in horse-power hours divided by the total input in water horse-power hours, the latter being actually computed from the discharge curves, which are not shown on the diagrams. In computing the average weighted efficiency for any given range of output, it is obviously necessary to use a definite load curve, and for purposes of comparison only a load curve has been used which varies from the minimum to the maximum output at a constant rate.

For example, if the load varies from 40 to 100%, the average weighted efficiency is about 89.5% for the 75 N_s wheel and about 84.5% for the 175 N_s wheel. If the load varies from 0 to 100%, the average weighted efficiency is about 87% for the 75 N_s wheel and about 80% for the 175 N_s wheel.

To obtain the correct efficiencies for any given station, the weighted averages must be computed according to the load curve which applies to the station. It should be noted that for most stations the comparison would be more favorable to the high-speed wheel than shown herein, because the plant would be operated for a greater length of time at high load than at low load.

The dotted curves are simply the arithmetical averages of the efficiency and are plotted to show that, at least for approximate preliminary purposes, where the load varies from about 25 to 100%, it is sufficiently accurate, perhaps, to use the more easily determined arithmetical average rather than the weighted average.

The possible exception to this statement is the high-speed turbine for the 2-unit plant, where the difference between the numerical and weighted efficiency is material. For the curves under discussion (Fig. 36), the numerical efficiency is always less than the weighted efficiency, but for the range where the curves are shown to be coincident, the differences are too small to be shown with the scale used.

Fig. 37 shows the same comparisons for a 4-unit plant, Fig. 38 for a 6-unit plant, and Fig. 39 for an 8-unit plant. It will be noted that with
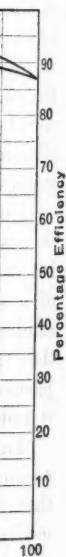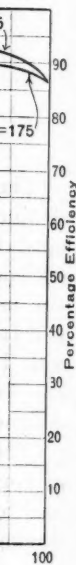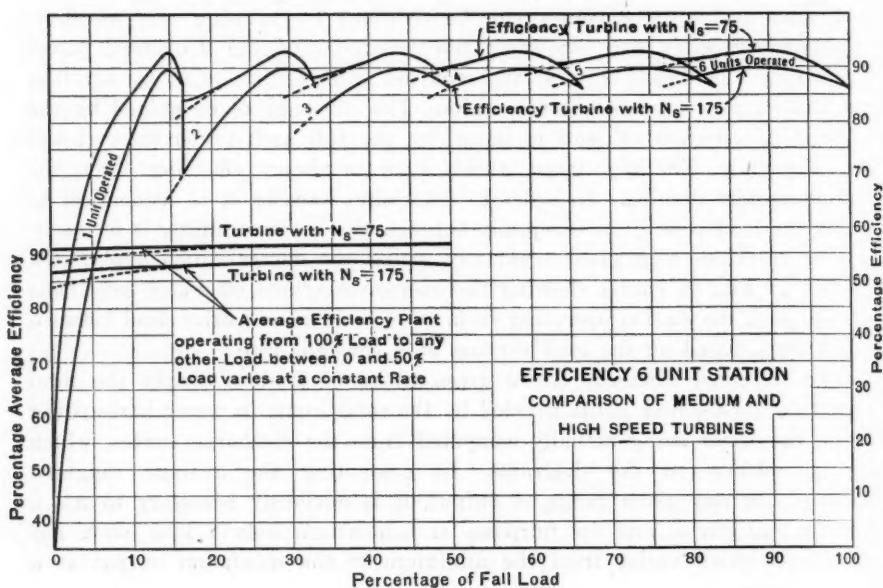
FIG. 38.
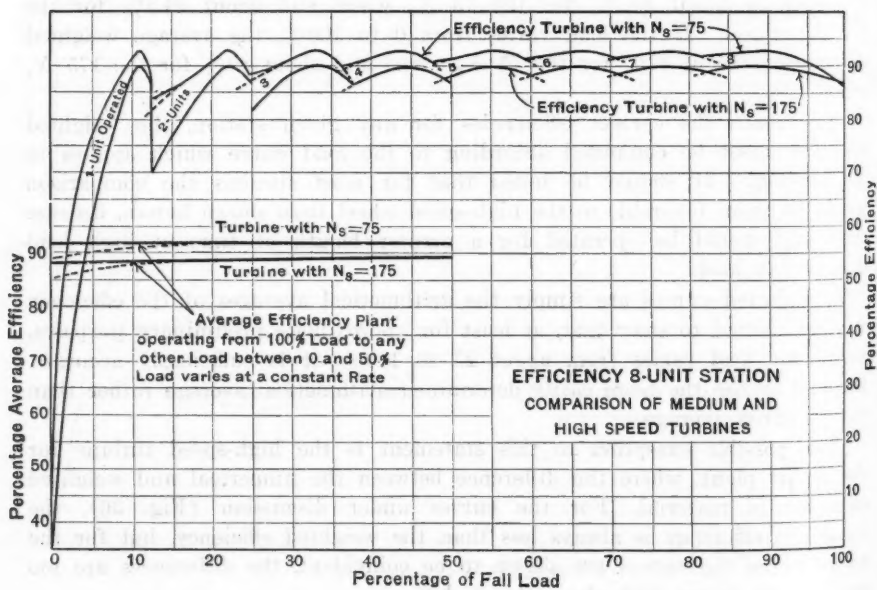
FIG. 39.

a 6 or 8-unit plant, medium speed, and an output equal to 50% or more of the maximum capacity, there is very little advantage in dividing the load other than equally between the units.

Fig. 40 shows how the average efficiency increases with the number of units. The full lines are plotted using the averages for ranges of load starting with full load and decreasing at a constant rate to any percentage between 25 and 50, the efficiency being practically constant for any range between these limits. The dotted lines are plotted using the averages for a range of load from full output to zero. The average efficiency for any number of units, with the range of load varying from full load to any point between 25% and zero, can easily be approximated.

COMPARISON OF MEDIUM AND HIGH SPEED TURBINES
Average Efficiency Turbines Operating from 25-50 to 100% Load
Average Efficiency Turbines Operating from 0 to 100% Load
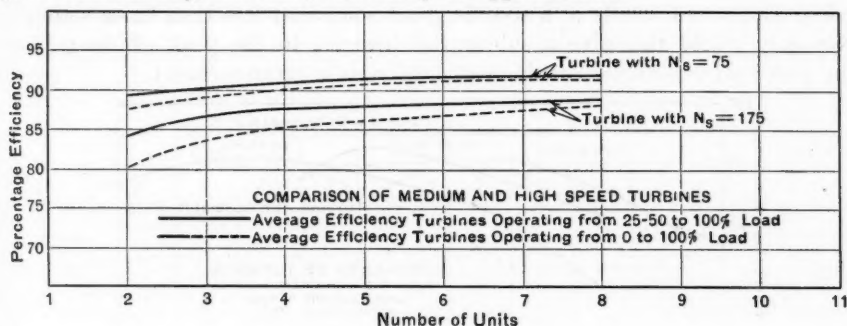
Number of Units
Fig. 40.

It is not intended that these curves be used to determine the average efficiency of medium or high-speed wheels; rather, the averages should be computed for each individual case. The curves simply indicate how the wheels compare in general. It must be considered that if the station is operated a greater length of time at nearly full capacity than at the low capacity, the average efficiencies will be increased over the values shown, and the averages of the high-speed wheels will increase more rapidly than those of the low-speed wheels.

RELATION OF EFFICIENCY AND SPECIFIC SPEED BASED ON TEST MODELS OF HOLYOKE SIZE
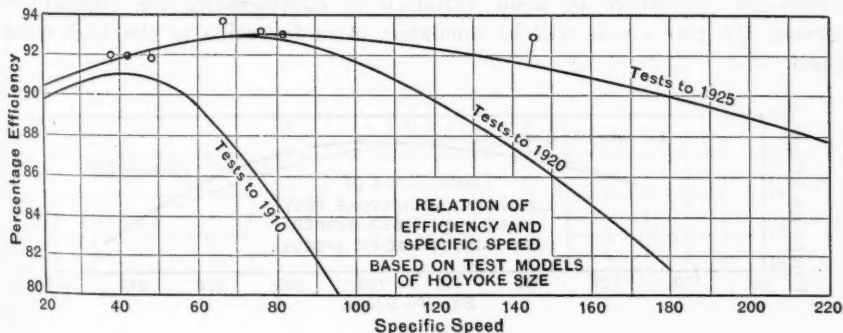
Specific Speed
Fig. 41.

Fig. 41 shows the "efficiency-specific speed" curves for 1910, 1920, and 1925, the efficiencies being based on results which can be obtained in the Holyoke flume, on wheels of the size usually tested. Undoubtedly, it is

possible to obtain somewhat better results on large wheels, with very good settings. It is interesting to note that little improvement has been made since 1910 in low specific speed wheels, and that the improvement since 1920 is entirely in wheels with specific speeds of 80 and higher. This comparison emphasizes the reasons for present interest in wheels of high specific speed.

The comparative average efficiencies, as shown in this paper, may be considered, perhaps, somewhat unfavorable to the high-speed wheel. To bring the average efficiency of the high-speed wheel to a point more nearly equal to that of the low-speed wheel, it is necessary to increase the efficiency of the former. An inspection of Fig. 41 will show that this can be done by using high-speed wheels of a specific speed somewhat less than those selected. Not only would this give a substantial increase in the peak efficiency, but the part-load efficiencies are increased to even a greater extent.
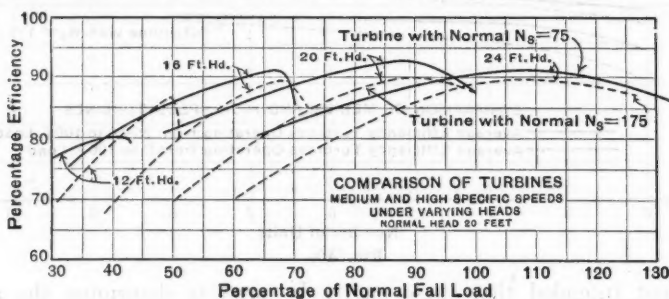
FIG. 42.

No attempt has been made to show a comparison of the average efficiencies under variable head conditions. Fig. 42 shows that the high-speed wheel has a better range at variable speed or variable head than the medium-speed turbine. The power and efficiency of the high-speed wheel hold up particularly well under low-head conditions. It follows, therefore, that where the percentage of head variation is considerable, the comparison between the two wheels will be somewhat more favorable to the high-speed wheel.
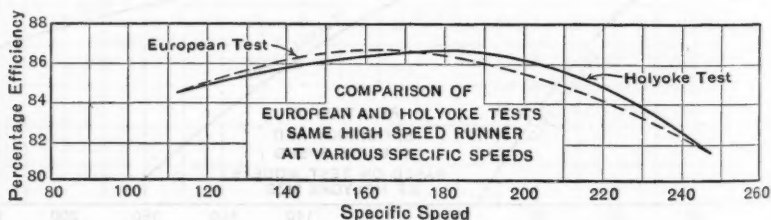
FIG. 43.

The comparison of European and Holyoke tests is a matter of general interest. Foreign engineers have always claimed that results obtained at Holyoke have been 2 or 3% higher than the correct actual. The writer has

recently had occasion to compare a Holyoke with a European test, using the same runner, draft-tube and wheel-case, and under the same head. Fig. 43 shows a comparison on the basis of the efficiency plotted against the specific speed. The difference between the tests is at no place equal to 1 per cent.
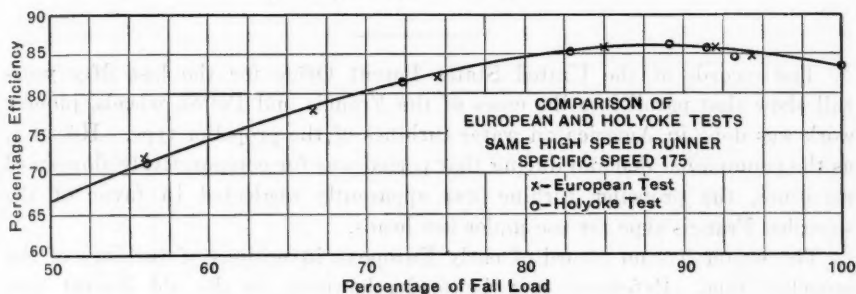
FIG. 44.

To show how the tests compare at the same speed, Fig. 44 has been plotted. The runner is about 19 in. in discharge diameter, and the efficiency differential between this size and one of normal size for the Holyoke flume is about $+ 3\frac{1}{2}$ per cent.

SOME APPLICATIONS OF THE
PROPELLER TYPE WATER TURBINE IN EUROPE

BY CHARLES C. EGBERT,* ESQ.

The records of the United States Patent Office for the last fifty years will show that parallel to the cases of the Francis and Pelton wheels, pioneer work was done in America on water turbines of the propeller type. However, as the commercial demand during that period was for comparatively slow-speed machines, the propeller turbine was apparently neglected in favor of the so-called Francis type for use under low heads.

The writer has no record of early European inventions of turbines of the propeller type. Reference may be made, however, to the old Jonval type turbine which was occasionally built with rimless runners, with the blades strongly bent, or curved, to suit the slow speed for which such turbines were designed.

It was not until about 1913 that the publication of the studies and experiments of Professor Viktor Kaplan, of Brünn, drew serious attention to the possibilities of the propeller type turbine. This has resulted in its development both in Europe and America.

The commercial application of the propeller turbine in Europe was un-doubtedly retarded by the World War; nevertheless, noteworthy progress has been made. This is indicated by the number of power plants in which various adaptations of wheels of the propeller type have been installed and the number of companies building that type. Furthermore, designs have been highly developed by intensive study by a number of manufacturers, coupled with much experimental work in numerous well-appointed testing flumes.

This paper makes no attempt to cover the entire subject of the application of the propeller turbine in Europe, and only gives brief descriptions of a few developments selected from data readily available.

The first to be referred to is the Lilla Edet Station (Fig. 1† and Figs. 45 to 48) built by the Swedish Government. This station contains three vertical shaft units, each of a nominal capacity of 10 000 h.p., at 62.5 rev. per min., under a normal head of 21.3 ft. (6.5 m.).

One of the turbine units is of the Kaplan type with a propeller wheel, 19 ft. (5.8 m.) in diameter, over the tips of four horizontally disposed blades (Figs. 46 and 48). It was built by the Verkstaden in Kristinehamn, Sweden. A most interesting feature of this unit is the use of movable runner blades which are pivoted in the runner hub so that the angle of the blades can be varied in accordance with the gate-opening and discharge of the turbine, thus insuring a flatter efficiency curve. The guaranteed output of this tur-

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* Cons. Engr., Niagara Falls, N. Y.
† See p. 1001.

bine under a head of 21.3 ft. (6.5 m.) is 11 200 h.p. (metric), from which a specific speed of about 144 ft-lb. units (637 metric) may be computed.

PLAN VIEW
LILLA EDET STATION
Scale in Meters
0 2 4 6 8 10
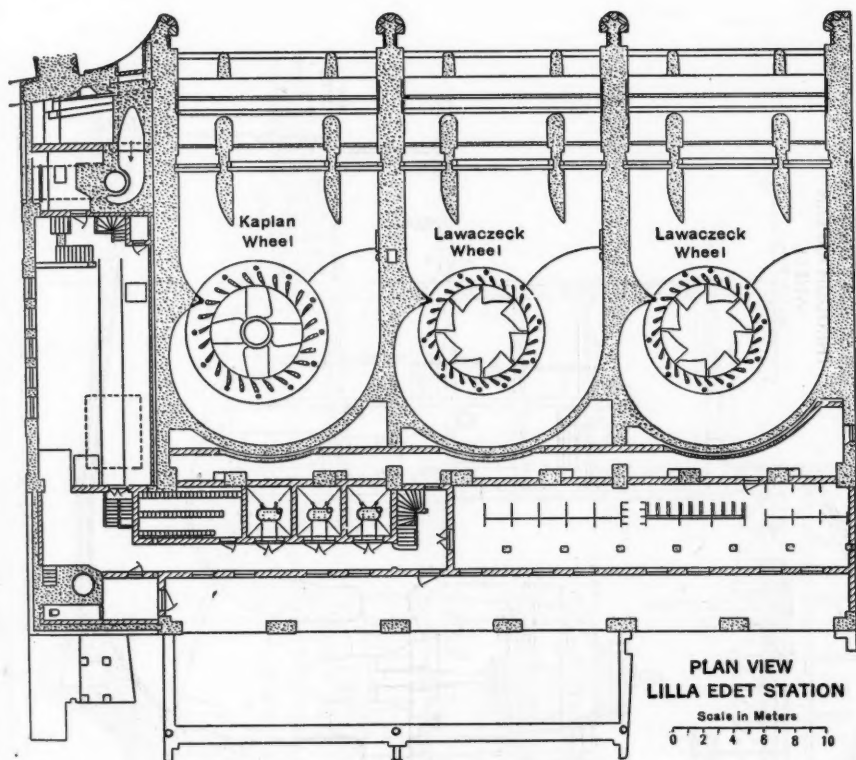
FIG. 45.

The other two turbines in the Lilla Edet Station are of the Lawaczeck type (see Figs. 1 and 3*), each having one runner with eight diagonally disposed blades, 19.6 ft. (6 m.) over the tips. These two units were built by the A-B Finshattan. The guaranteed output of each of the Lawaczeck turbines under a head of 21.3 ft. (6.5 m.), is 10 400 h.p. (metric), from which a specific speed of about 139 ft-lb. units (616 metric) is computed.

The final designs of the Lilla Edet turbines, including wheel-cases and draft-tubes, were evolved after much experimentation in testing flumes, using model runners up to 3.27 ft. (1 m.) in diameter. Various well-known forms of draft-tubes were tested before the decision to use the elbow type was reached. Fig. 4† gives the performance of the turbines at Lilla Edet.

Tables 1‡ and 3 show the relative capacities of some large single-runner turbines, which have been installed in different countries.

* See p. 1002.
† See p. 1003.
‡ See p. 1004.

FIG. 46.

FIG. 47.

LONGITUDINAL SECTION
LILLA EDET STATION

Scale in Meters
0 2 4 6 8 10

CONNECTIONS FOR
HYDRAULICALLY OPERATING
THE KAPLAN RUNNER BLADES
AT LILLA EDET
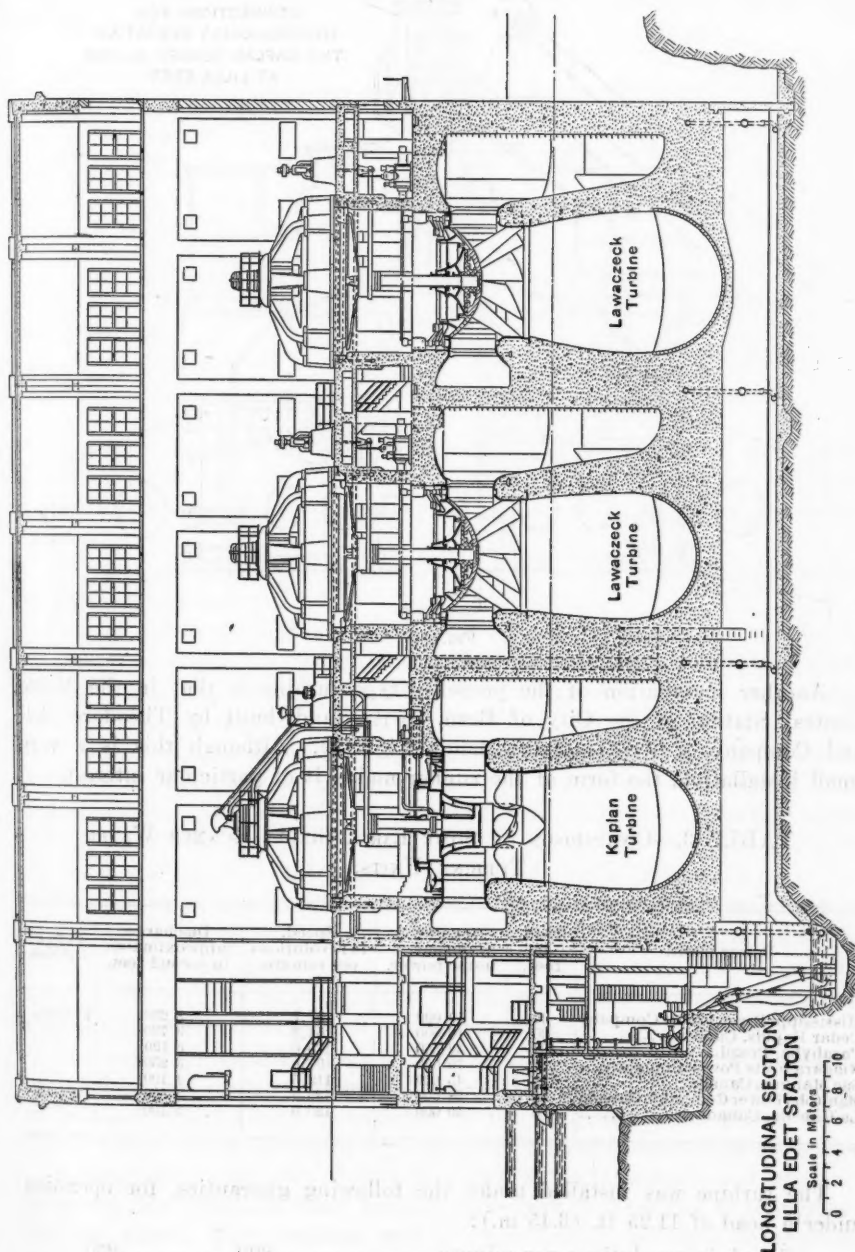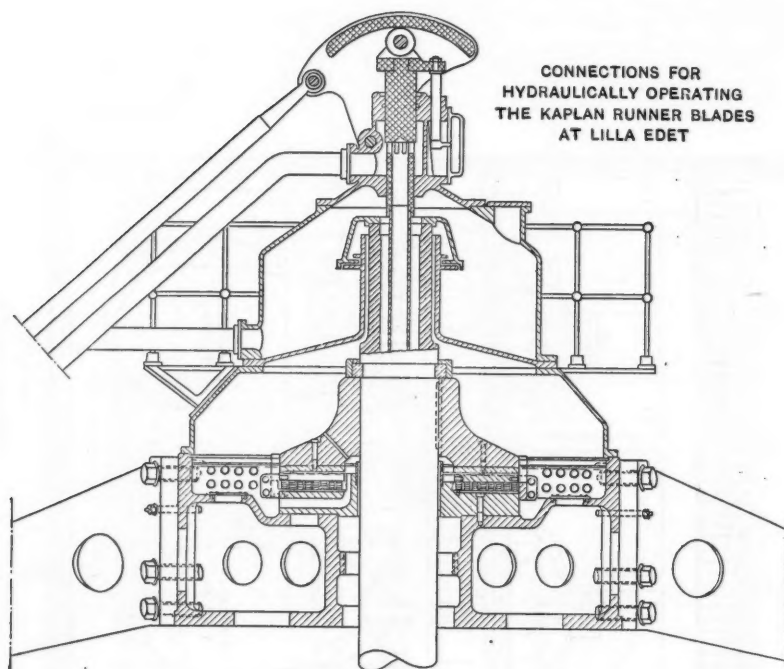
Fig. 48.

Another installation of the propeller type turbine is that in the Matte Central Station of the City of Bern, Switzerland, built by Theodore Bell and Company of Kriens, Switzerland (Fig. 49). Although this is a very small installation, the form of the runner makes it of particular interest.

TABLE 3.—COMPARISON OF SOME LARGE SINGLE-RUNNER WATER TURBINE UNITS.

Power plant.	Head, in feet.	Generator output, horse-power.	Speed, in revolutions per minute.	Discharge (approximate), in second-feet.	Type of turbine.
Mississippi River Power Company	32	10 000	57.7	3 230	Francis
Cedar Rapids, Canada............	30	10 800	54.3	3 730	"
Parahyba, Brazil.................	104	32 500	125.0	3 120	"
Niagara Falls Power Company...	213	70 000	107.0	3 250	"
Isle Maligne, Canada.............	110	45 000	112.5	4 100	"
Manitoba Power Company, Canada	56	28 000	138.5	5 100	Propeller
La Gabelle, Canada..............	60	30 000	120.0	5 100	"

The turbine was installed under the following guaranties, for operation under a head of 11.25 ft. (3.45 m.):

Speed, in revolutions per minute..............200..........250
Output, in horse-power.......................290..........323
Percentage of efficiency...................... 80.......... 78

From this, the specific speed may be computed as about 165 ft-lb. units (725 metric), at 200 rev. per min., and 216 ft-lb. units (955 metric), at 250 rev. per min.

PLAN OF
MATTE TURBINE   (a)   (b)   VERTICAL SECTION
THROUGH
MATTE TURBINE
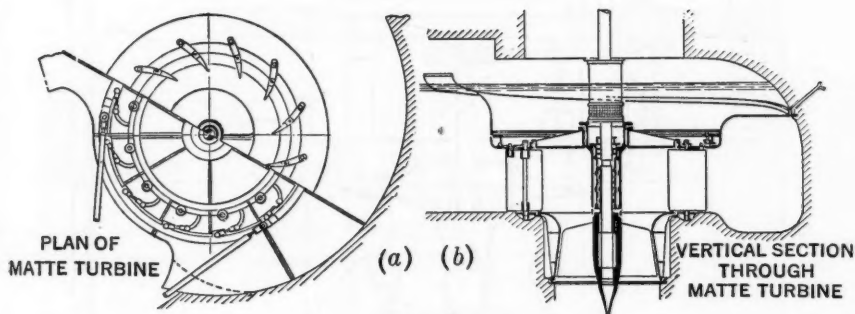
FIG. 49.

The form of the runner is interesting; it is a two-bladed screw type, each blade covering about 180° of the circumference. The final design of the unit followed extensive experiments in the testing flume of the manufacturer.

The construction of the Matte turbine and its efficiency curve are shown by Figs. 8*, 10†, and 49.

One of the latest installations is that of the Wynau Electricity Works on the Aare River, in Switzerland. This installation includes two units, each of which has a nominal capacity of 2 700 h.p. and operates at 107 rev. per min. under a variable head of 8.2 ft. (2.5 m.) to 17 ft. (5.2 m.). These units were furnished by the Ateliers de Construction Mecaniques de Vevey. Acceptance tests of these turbines indicate a maximum efficiency of 89%, with a specific speed of 165 ft-lb. units (728 metric), when operating under a head of 17 ft. (5.2 m.). Under a head of 8.2 ft. (2.5 m.), at 85% efficiency, the specific speed is given as 195 ft-lb. units (862 metric).

The design of the turbine runner was developed with the assistance of tests of models 15 in. (380 mm.) in diameter. The generator coupled to the Wynau turbine is designed for a runway speed of 2.7 times the normal speed of 107 rev. per min. In Figs. 50, 51, and 55 are shown details of the construction and performance of the Wynau turbine.

The writer particularly regrets that he is not in a position to present the developments made by Escher Wyss and Company, of Zurich, Switzerland, which have followed very remarkable results obtained in its testing flume. Fig. 7* shows one of this Company's propeller type runners having six blades.

A reluctance on the part of European turbine manufacturers and engineers to publish the results of their findings in respect to propeller type turbines, has been noted. This, perhaps, may be explained by the rapid development accomplished, leaving little opportunity to pause and review the results.

* See p. 1005.
† See p. 1007.

VERTICAL SECTION OF
THE WYNAU TURBINE

Fig. 50.

CHARACTERISTIC CURVES OF
THE WYNAU TURBINE
Constant Speed =107 Rev. per Min.

Specific Speed Ns

Full Power

7/8 Power

6/8 Power

Percentage Efficiencies η

Maximum Discharge

Specific Speed

Horse Power

Quantity of Water
in Cu. Meters per Sec.

Net Head H, in Meters

Fig. 51.

TESTING FLUME AT
LILLA EDET, SWEDEN
Scale in Meters

FIG. 52.

TESTING FLUME OF TH. BELL AND CO.,
KRIENS, SWITZERLAND

FIG. 53.
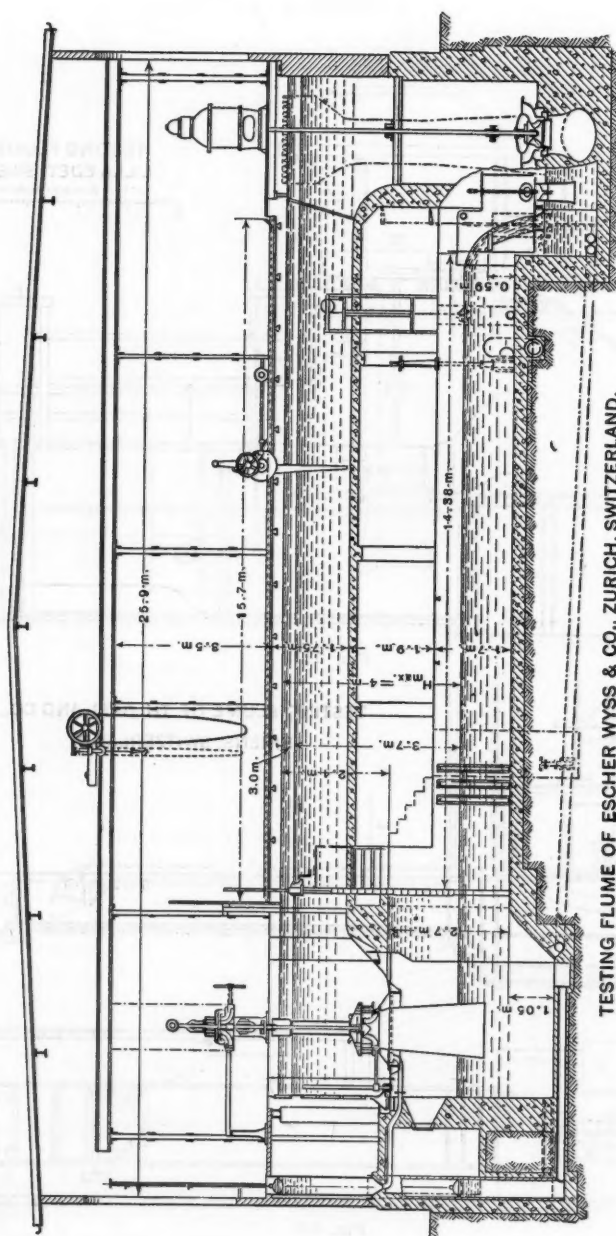
TESTING FLUME OF ESCHER WYSS & CO., ZURICH, SWITZERLAND.
LONGITUDINAL SECTION
FIG. 54.
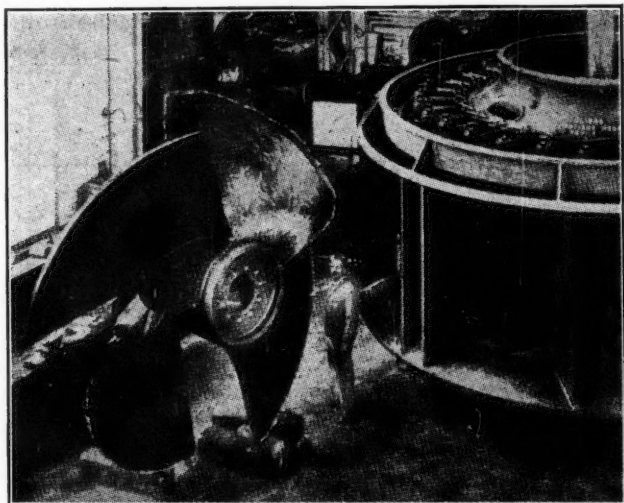
FIG. 55.—VIEW OF RUNNER AND GUIDE APPARATUS
OF THE WYNAU TURBINE
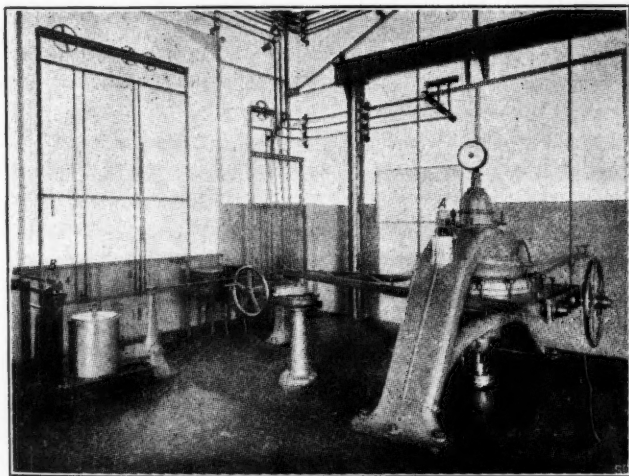
FIG. 56.—OPERATING ROOM FOR TESTING FLUME OF ESCHER
WYSS AND COMPANY, ZURICH, SWITZERLAND.

Fig. 52.—View of Intake and Drum Apparatus
of the Wigen Turbine.

Fig. 53.—Installation View for Cooling Plant of Wigen
Type and Turbine, General Switzerland.

In the descriptions given, references to the testing of model runners in the development of the turbine designs have been made, which testing, it is easy to believe, is very important in respect to the propeller type runner. It appears, therefore, not without the scope of this paper to show Figs. 52, 53, 54, and 56, illustrating some of the flumes used in testing models of the turbine runners referred to by the writer.

PROPORTIONING OF UNITS IN LOW-HEAD PLANTS

By John P. Hogan,* M. Am. Soc. C. E.

In a paper delivered before the American Society of Civil Engineers on October 4, 1922,† the writer indicated certain tendencies in present water power development, the more important of which were developments in storage and interconnection and a tendency toward increase in the size of units.

Increased interconnection has accomplished two important results: First, by broadening the market, it has made it possible to consider the distribution in the general market of the output of the large continuous powers of boundary streams; and, second, it has permitted the selective development for peak purposes of plants on interior streams, by removing the need for flexibility in any individual plant.

Storage on interior streams, now in process of construction or proposed, will permit not only the selective development of individual plants but will make possible the unified development of entire streams for peak power capacity.

If it is assumed that the average present installation on the interior streams is about equal to the average flow and that such a stream may be completely regulated so that the minimum flow will be brought up to the average flow, a development of this stream for 8-hour peak power will require three times the present installation. The development of regulated streams for peak power capacity requires that the time element between plants be eliminated, if possible, in order that the peak capacity in all the plants may be synchronous with the peak demand. This requires a continuous chain of plants with sufficient capacity in each pond to correct the time of travel of the water from the preceding pond. In order to make the chain continuous it is necessary on the average stream to develop relatively low-head plants which might not otherwise be economical.

In the plans for development of boundary streams a number of relatively low-head plants have been proposed. At the same time the uniformity of both the continuous and regulated flows insures the operation of larger units at full capacity and best efficiency. There is a rapidly increasing demand, therefore, for larger capacity units operating under relatively low heads.

In proportioning units for a plant, the controlling factors in determining the number of units, assuming load regulation to be provided from other sources, are: Maximum head and variation in head; character of the stream flow; speed of units; efficiency; and velocity of approach and discharge of water.

The writer does not propose to discuss in detail the variations in head and flow as they represent special problems to be considered and determined for each particular case.

For ordinary service on a fluctuating flow and varying load the Francis wheel with its relatively flat efficiency curve at varying flows is the ideal. The

* Cons. Engr., New York, N. Y.

limit of size in Francis wheels for low heads is soon reached, due to decrease of speed in proportion to the increase in size.

For a 30-ft. head the normal Francis runner operating at full efficiency should have a specific speed of about 90. For a 30-ft. head and specific speed of 90 the characteristics of normal Francis wheels of various sizes would be approximately as shown in Table 4.

TABLE 4.

Quantity of water, in cubic feet per second.	Revolutions per minute.	Electrical horse-power.
500	170	1 330
1 000	125	2 660
2 000	85	5 300
4 000	55	10 800

At the largest size shown, the size and cost of the wheel and generator become almost prohibitive. It will be readily seen therefore that speed fixes a definite limit to the economic size of Francis wheels for low-head developments. Various experiments have been made with special designs and modifications of Francis wheels to exceed the normal specific speeds, but, in the writer's opinion, they do not promise a solution of the problem.

As has been stated, designers and manufacturers, both in the United States and abroad, have put at the disposal of the constructor and operator special types of wheels to meet this situation, which heretofore have been called propeller wheels. These wheels have somewhat different designs and different spheres of usefulness, but they have the common object and result of increasing the specific speed at a slight loss in maximum efficiency.

The result is increase in speed of wheel and generator for a given quantity of water and consequent decrease in cost of wheel and generator or a decrease in the number of units, all resulting in power-house savings. A normal propeller wheel operating on a 30-ft. head will probably take 4 000 cu. ft. per sec. of water at 85 rev. per min. and therefore would be roughly equivalent to the 2 000 cu. ft. per sec. Francis wheel operating at the same speed although the propeller wheel will develop nearly twice the power.

It would appear also that the limit of size of units for low heads has been substantially doubled, if 50 rev. per min. is considered to be a reasonable minimum speed for both types. It is true that all the so-called propeller wheels, except the type with movable blades, have increasingly lower relative efficiencies at part gates than the Francis wheels. This is of no importance in continuous plants of large flow where the number of units is so great that sufficient flexibility can be obtained by combinations of units. On stream-flow plants, on the other hand, it is probable that at low flows the combinations of propeller wheels of large flow might give low efficiency and capacity at the most critical periods and that economy will dictate the use of two types of wheels of which the majority will be propellers and the minority either Francis wheels, or special wheels having characteristics similar to those of the Kaplan

wheel. This has already been done abroad for a low-head plant, as at Lilla Edet, Sweden.

In peak capacity plants it will be easier to arrive at a compromise in the number of units which at the same time will permit a large wheel and efficiency at relatively small daily flows, since it will be possible to operate for a relatively short time on a concentrated flow.

Other characteristics of the propeller type which should be considered are that they give better efficiencies at fluctuating heads and that they must be set much lower than wheels of the Francis type.

This leaves only the velocity of approach in the penstock and velocity of discharge to the tail-race to be considered. The limitations imposed by the propeller type are very similar to those of the Francis type and in low-head plants of large flow these limitations are already presenting serious problems in the design and construction of waterways.

Already, under some conditions, the provisions of adequate water passages are of more influence in limiting the size of the units than the characteristics of the wheels, and with increasing use of wheels of the propeller type in large units, the influence of these hydraulic limitations will increase. They may even be finally responsible for placing a definite limit to the increase in size of units for plants of large flow and relatively low head.

THE ENGINEER AS A RAILROAD EXECUTIVE*

By Julius Kruttschnitt,† Esq.

To correctly judge of the qualifications of a man for the position of "Railroad Executive", a comprehensive survey of the duties devolving on the chief executive officer of a railroad under present-day conditions and of the difficulties which he has to overcome, must be made. Not only must the problems that require solution be considered but the limitations imposed by law on the revenues of railroads and the extent of control of their expenses that the executive is permitted to exercise. It seems appropriate, therefore, to introduce a quite full outline of the problems that confront the responsible managing officer, to show the extent to which so far he has solved them, and to what extent they remain unsolved, and then to show how the engineer has risen to new responsibilities, and what recognition stockholders and directors of railroads have accorded to his fitness to administer their properties.

Fundamentally, a railroad is a huge manufacturing plant designed to convert the energy locked up in fuel into work for transporting persons and property on specially designed roadways. The energy conversion is by means of individually operated locomotive units, and the success of the entire plant, measured by the spread between production costs of output, measured in passenger-miles and ton-miles, and in selling prices, depends on the efficient working of each individual unit.

To attain the maximum or even moderate success in creating a margin between the cost and sales prices of output, the necessary executive and administrative work can best be done by men specially trained in those branches of knowledge that contribute most to the efficient operation of railroads, on the degree of which depends success or failure; and the most important duty of the chief executive is to so select his official staff as to attain the best results. To pass intelligently on the qualifications of his staff and their recommendations that may reach him—in other words, to avoid being a mere "rubber stamp" to approve whatever is laid before him—requires a fair knowledge of the functions of each department head by the chief executive.

At the outset, the speaker will explain the causes that have brought about the epochal changes in the conditions of the railroads in the first quarter of the Twentieth Century, conditions that have obstructed successful operation to an extent never theretofore encountered, and then show how American

talent has adapted the railroads to these new conditions and so far has averted bankruptcy and ruin.

From Fig. 1 it will be observed that the rise of prices of all commodities throughout the country, started by the outbreak of the World War in the summer of 1914, culminated in the United States in May, 1920, when the index number of wholesale prices showed an increase of 236% above the

FIG. 1.—RELATION OF PRICES, RAILWAY WAGES, FREIGHT RATES, AND RETURNS ON INVESTMENT FOR TWENTY-FIVE YEARS COMPARED WITH DATA FOR 1900.
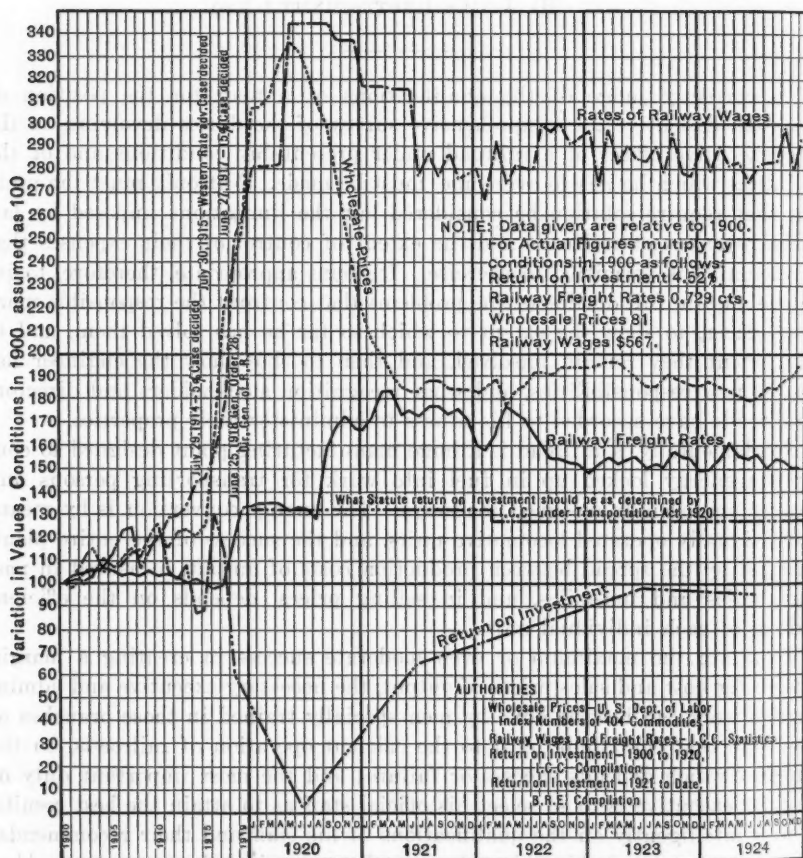
corresponding number in 1900. Starting from the base index number—taken as 100 in 1900—there was a slight fall in 1901, but in 1902 prices were 4% above those of 1900 and, with some reversals, the rise was continuous to the outbreak of the World War, when they were 20% higher than in 1900. At the date of America's entry into the war, the index number of wholesale prices was 120% above that of 1900. The fall from the peak of 236% over 1900 in 1920 was rapid to 83% above 1900 in the middle of 1921, since when there have been fluctuations leading to the figure of 94% above 1900 in December, 1924.

During the 17 years from 1900 to 1917, the average railway freight rate in the United States remained nearly stationary. True, there were fluctuations that affected the average but never sufficiently to raise it over 5 to 7% above 1900, so that when the United States entered the World War the average freight rate was almost exactly the same as it was 17 years theretofore, but in 17 years average prices had risen 120% and wages of railway employees had risen 77 per cent. At the beginning of the century, 25 years ago, the average operating ratio for all the roads of the United States was 65 per cent. Under Government operation it rose in 1920 to 94.3% and for the twelve months of 1924, after four years of private operation, it fell to 76.14 per cent. These data are quoted to show how the spread between revenues and expenses has shrunk, and how narrow a margin exists at the present time out of which to pay interest on obligations and dividends on stocks.

It will be observed that the percentage return on investment that was 4.52 in 1900 substantially vanished in 1920 on the return of the railroads to their owners by the Government, and never since that time has equalled the 1900 return. For 1924 it was only 96% thereof, or 4.35%; since then it has fallen still farther below the statutory return prescribed by Congress as reasonable and allowable, 6% from March, 1920, to March, 1922, and 5.75% from March, 1922, to date.

The figures quoted show the wide difference in the kind of managing talent required on railroads now and 25 years ago. At the end of the Nineteenth Century, 35 cents net out of every dollar could be turned into the Treasury, while at the present time only 24 cents, or 32% less, can be turned in. It requires the closest kind of management and the services of very much more highly trained executives to meet all money requirements other than mere operating expenses with only 24 cents out of every dollar. To get the most work out of fuel, the largest material item of operating expenses, taxes the efforts of substantially every department on a railroad, and to obtain the best results requires the highest talent in a great many different branches of engineering. The course of Civil Engineering in leading universities embraces courses in the allied sciences of Mechanical and Electrical Engineering, and the important principles of physics, chemistry, geology, and economics, so that the executive who has enjoyed the training afforded in a civil engineering course is peculiarly well fitted for co-ordinating and guiding the numerous operating activities on railroads and for securing the maximum output of salable products at minimum costs. An analysis of operations that produce transportation units will show that the engineer who has been trained to secure desired ends in the most practical, efficient, and least expensive way can best contribute to the success of a railroad enterprise, and will indicate the directions in which he should prepare himself in college for executive duties, and will correspondingly indicate to the owners of railroads, other things being equal, the qualifications to seek in managers.

To repeat—viewing the railroad as a manufacturing plant designed for the sole purpose of creating the maximum of transportation units out of the minimum of fuel units, the greatest efficiency will be attained by that railroad whose executives co-ordinate best all departments and concentrate their activities

on increasing output, from the sale of which the railroad must secure its revenues.

Enumerating some of the activities of a railroad will show how numerous are the demands made on a chief executive, and at the same time will show to a certain extent how the problems that confront him have been partly solved or are still demanding study for their solution.

I.—EXECUTIVE DEPARTMENT

The wise executive will spare no expense in establishing an Accounting Department to keep track of earnings and expenses; to record the history of operations; to act as general inspectors; to expose poor results as prerequisites to their correction; to work up promptly cost data of every important item of operating expense and as promptly to put them in the hands of all concerned. While the experience of the men who make the decisions will usually keep them from going far wrong, yet in matters involving millions of dollars in both capital and operating expenses, a railroad officer should not be satisfied unless he has definite figures on which to base decisions. The record traffic handled with record efficiency in 1917 was the response of the railroads to the monthly data sheets first issued by the Railroads' War Board; these are now issued in even more detail by the Interstate Commerce Commission and the Bureau of Railway Economics. Fuel accounts kept with individual engineers and firemen and locomotives furnish railroads with data from which to decide definitely what type of locomotive is the most economical for their use, and whether the expense of applying and maintaining some of the many fuel-saving devices is warranted.

"The location of a railroad is giving it its Constitution. It may be sick almost unto death with accidents of construction and management, but with a good Constitution it will ultimately recover." The board of directors, advised by the chief executive, must pronounce on the location of the railroad between its termini; it allots the amount of money to be spent on construction; fixes the grade and curve systems, because the ideally perfect railroad, between given points, should be absolutely straight and of uniform grade, that is, devoid of undulations in alignment or profile. The expense of operation will increase in proportion to the grade and curve resistance involved in departures from the ideal line, and the extent of departure must be authorized by the board, on which devolves the duty of providing the money for construction. Whenever a ton weight is lifted 1 ft., an amount of work equivalent to moving it 333 ft. over a straight, level track is spent; and every time a ton weight of train is forced through 100 ft. of 1° curve, the work done in deflecting it from a straight to a curved path is equivalent to moving the same ton over $16\frac{3}{4}$ ft. of straight, level track. The equated length of the old line of the Central Pacific between Ogden and Lucin, Utah, compared with the new Lucin Cut-Off line is as 280 to 129 miles; 1 lb. of fuel on the new line moves as much tonnage as 2.2 lb. on the old one.

The Executive Department must determine the justifiable outlay to reduce grades and curvatures on existing lines, as obviously the expenditure necessary to obtain a straight line between two points without any undulations in grade

might be commercially prohibitive. It is, therefore, of the greatest importance for the commercial success of a railroad to balance the money cost of approaching as nearly as possible ideal conditions with the expected benefits.

Motive power can often be economized by expensive physical improvements, such as installing automatic block signals, re-designing terminals, modernizing roundhouses so as to utilize the residual heat from incoming locomotives to raise steam on outgoing ones, and increasing the number and lengths of sidings to save delays on the line. On the chief executive rests the responsibility of balancing the interest, maintenance, and depreciation costs on large capital expenditures against possible savings, and of providing the necessary capital if the investment is justified.

II.—Traffic Department

The Traffic Department is the sales department of the railroad, and its contract specifications as to loads in cars and character of service exert a profound influence on net earnings. It requires 41% less power to haul 50 tons of lading in one car than in two cars of the same class and weight, and 53% more power to haul a given load at 40 than at 20 miles per hour.

III.—Maintenance of Way Department

The resistance to traction of locomotives and cars varies between wide limits with the condition of the track. The train resistance figure commonly used of 6 lb. per ton (2 000 lb.) at low speed assumes track in good condition. It may very easily be two or three times that amount on track laid with light rail, poorly maintained, in bad surface, and with low joints, etc. The heavier the rail and the better the ballast, the easier it is to maintain good alignment and surface, the less will be the deflection under loads, and the less will be the work required to force the wheels out of the depression caused by weak and yielding joints.

IV.—Maintenance of Equipment Department

About one-fifth of all locomotive coal (in 1921 this amounted to 25 000 000 tons) was consumed when the locomotive was not doing useful work: in firing up, in waiting for trains, in standing on sidings and passing tracks, in delays at terminals, drawing fire, and waiting at ash-pits. This consumption is not productive, nor is it all waste. By re-designing terminals and increasing the lengths and numbers of sidings the waste can be minimized. Ingenious graphs on which obstructions to traffic are likened to obstructions to the flow of water in incrusted pipes have proved of great assistance to indicate how to increase the capacity of the line and defer double-tracking.

In designing locomotives, the features exercising the greatest influence on fuel efficiency are: Careful and expert proportioning of engines, boilers, fireboxes, grates, flues, draft appliances, so as to produce by their combination the most perfect device to extract from fuels their full heat contents. Barely 8% of the energy in fuels is now extracted and converted into work.

While it is customary to speak of the waste of fuel in the average locomotive, the extremely unfavorable conditions under which it is required to

work must not be forgotten. Space limitations have thus far prevented the successful use of condensing engines, and this has deprived the locomotive of the advantage of 12 to 13 lb. additional effective cylinder pressure. It is driven through all kinds of weather at high speed, so that radiation losses are excessive. To meet the demands made on its boiler for steam, explosive draft must be used, which expels much unburned fuel from the firebox; the resultant effect of all this is evidenced in low thermal efficiency.

With the increase in the power of locomotives and the use of higher steam pressures, mathematical talent instead of "rule-of-thumb" has been called on to compute stresses and to proportion parts. Questions as to the adoption of alleged improvements are difficult to decide, as they involve first cost, interest, maintenance and depreciation, their possible effect on engine failures and the idle time of locomotives while undergoing repairs. Sometimes economy of operation is sacrificed to facilitate adjustment and repairs, as when outside valve gears replaced the original Stephenson link motion.

The practice of buying locomotives regardless of their adaptability to special service, as was done during Federal control, simply because they could be built in quantity at some reduction in cost, is recognized by competent judges as the poorest kind of economy.

The brake thermal efficiency—or the percentage of energy in the fuel transformed into work—for various engines is as follows:

> Reciprocating, condensing, stationary steam engines
>   (boilers excluded) 19.0%
> Steam turbines 19.2%
> Average of eight large power plants (boilers included)... 13.3%
> Non-condensing steam locomotives at maximum capacity,
>   equipped with proven fuel-economizing devices. 7.3 to 8.1%

Some of the problems that have been closely studied and have been solved to a greater or less extent are:

Brick Arches.—Brick arches built into the firebox mix the entering air thoroughly with the combustible gases rising from the fire and save on some types of locomotives about 12 per cent.

Superheating.—The advantages resulting from the use of superheated steam are due mainly to:

> (a) Prevention of cylinder condensation; and
> (b) Larger volume of steam per pound.

Since the cylinder takes the same volume of steam per stroke for the same cut-off, it is obvious that substantially less pounds of steam will be used for the same work done. Superheating may save 20% of the fuel.

Feed-Water Heaters.—Feed-water heaters utilize waste heat to raise the temperature of the feed water, thereby relieving the fuel in the firebox from heating the cold feed water from an average of 60° Fahr. up to 220° Fahr. and saving about 10 to 14% of the fuel.

The combined average effect of these three devices in saving fuel on passenger and freight locomotives of various types and speeds in actual service is:

Arch .... 9 %
Superheater .... 20 %
Feed-water heater.................................. 10 %
Combined 34.5%

These figures vary widely for different locomotives, speeds, and classes of service.

The savings claimed for the various fuel-saving devices are gross figures. The cost of application is quite large, and in some cases not justified by the age or obsolescence of the locomotive; in others, it is beyond the financial ability of railroads to apply them; hence care must be taken to balance the cost of interest, maintenance, and depreciation against the value of the fuel saved. The net saving so determined—not the gross—will show whether or not the money to provide the appliances can be providently expended. Experience shows that thus tested the gross saving of superheaters is reduced from 20 to about 17.4%; of feed-water heaters from 10 to about 6 per cent.

Boosters.—Under normal conditions a locomotive can haul as much, or even more, load than it can start. The booster consists of a pair of independent engines coupled to the trailer wheels of locomotives, thus utilizing weight not otherwise used for traction, in starting and quickly accelerating trains and in reducing violent shocks in starting, which are so hard on draft gear and the temper of passengers. As its power cuts out automatically at a speed of 10 miles per hour, it uses steam at low speeds when the boiler can over-supply the main cylinders. It is, therefore, an indirect but valuable fuel saver through increased locomotive capacity. The booster can add about 10% to the starting drawbar pull of heavy freight and as much as 25 to 35% to the starting drawbar pull of lighter passenger locomotives.

Feed-Water Purifiers.—After selecting the softest natural feed waters, such others as may have to be used should be chemically treated to reduce objectionable impurities to a minimum. A conservative estimate of the effect of scale $\frac{1}{16}$ in. thick on flues is to increase fuel consumption 10 per cent. The Hungarian State Railroads on 3 000 locomotives use a mechanical feed-water purifier mounted on top of the boiler. The water on entering the purifier is subjected to full boiler pressure and temperature, and deposits its scale in and on this adjunct to the boiler instead of in the boiler and on the flues. With reduction of scale there is a direct saving in fuel consumption, while a reduction in frequency of boiler washings saves considerable time and fuel. Without the purifier, locomotives were washed on an average every 5 days; with the purifier, the required washing was once every 54 days.

Lubrication.—All journal bearings must be carefully lubricated to reduce friction; so must the center and side-bearings of cars, the friction of which prevents the adjustment of trucks on entering and leaving curves and thereby, in the aggregate, causes a heavy increase in the resistance to traction that the locomotive must overcome.

Wear on all wheels of locomotives, tenders, and cars must be watched. If the treads are worn hollow or flanges worn vertical, the areas in contact with the rails when cars are oscillating or rounding curves will be increased and the friction correspondingly increased. A heavy freight locomotive, tender, caboose, and sixty freight cars are carried on more than 500 wheels; a slight increase in the friction on a few of them only would, in the aggregate, create a heavy drag on the locomotive.

Flange Lubrication.—Flange lubrication reduces friction between rail and flanges and, therefore, has a substantial effect in reducing fuel consumption. On two roads where flange lubrication was used, locomotive tires showed average mileages between turnings of 33 000 and 44 000, respectively, compared with 16 000 and 23 000 without such lubrication. Similarly the wear on rails in curves showed a reduction of as high as 87 per cent. On one road of excessive curvature and grade the life of locomotive tires was increased. 375% with flange lubrication.

Fuel Specifications.—To insure full heat value the specifications for fuels purchased should cover their value in British thermal units.

Maximum Loads in Vehicles.—Freight cars should be fully loaded as far as may be. The resistance of a train of empty freight cars may run up to 8 lb. per ton; that of cars with average loads to 6 lb. per ton, while that of heavily loaded cars, of total weight, 60 to 70 tons, may be as low as 4 lb. per ton. The load, in other words, should be carried in as few cars as possible.

Reduction of Dead Weight.—As the expenditure of fuel in hauling a ton is the same whether or not it is paying freight, it is evident that the smaller the percentage of non-paying or dead weight to total weight moved, the smaller will be the cost of hauling the paying freight.

The United States Railroad Administration's 50-ton, single-sheathed box car weighs 47 200 lb.; the American Railway Association's 50-ton box-car design weighs 42 400 lb. Thus, the excess weight of the United States Railroad Administration's car is 4 800 lb., or 11.3 per cent.

Based on 1923 data, the saving in fuel by eliminating this excess weight, is $9.77 per car per year, or $4.10 per ton, which, capitalized at 5%, shows that an added initial expense of $82 in building a car is justified to reduce its weight 1 ton.

As far as practicable all weight in a locomotive above what is needed for friction to develop traction should be eliminated.

Boiler design to suit the class of service, and the character of the fuel, requires considerable flexibility in this principle.

Stimulation of Interest and Co-Operation in Saving Fuel.—Assuming that locomotives are properly maintained, in order to obtain the best results from burning fuel it is necessary to inspire the interest and secure the co-operation of the men on the foot-boards of 65 000 traveling power plants. How these aids are attained will appear from a brief of the practice on a large system covering many States and operated under the most varied topographical and climatic conditions:

1.—In addition to the inspection at each terminal of engines on their arrival and again before departure to insure the correction of all defects before the engine begins its trip, very thorough monthly inspections are made of all parts of the engines, boiler and appliances, draft arrangements, etc.

2.—Classified statistics supply a complete history of operation. To be of most use they are issued immediately after the occurrences they record:

> (a) A fuel bureau receives the report of all train and locomotive movements within two days. Ton-miles, fuel consumed per 1 000 ton-miles, tons per train for each individual locomotive and each individual engineer and fireman are then computed, and the results put into the superintendent's hands three to six days after the movement, so that shortcomings can be detected and quickly corrected.
>
> (b) Monthly statements of individual performance of engineers, firemen, and locomotives are displayed on bulletin boards.
>
> (c) A sheet recapitulating results by divisions is issued monthly.

3.—On each division a fuel committee composed of four engineers, four firemen, one conductor and one brakeman, serves for six months and holds meetings bi-monthly, the superintendent acting as chairman. The meetings are well advertised and are attended by the superintendent's staff and employees from all related departments, who offer suggestions freely. Experts on operating methods and appliances deliver addresses. Motion pictures illustrating actual road conditions are exhibited. Minutes of meetings are widely distributed and published in an employees' magazine.

4.—To create emulation between divisions, a red silk banner is awarded quarterly to divisions showing the greatest improvement in the previous year.

5.—Gold-plated cap badges are awarded quarterly on each division to the engineer and fireman showing the best fuel record in through and local freight and passenger service.

6.—Each superintendent publishes the names of five engineers and five firemen having the best fuel records for the month in through and local freight, passenger, and yard service on a roll of honor, which is posted in roundhouses and printed in an employees' magazine. A letter of commendation is sent to each man whose name appears on the roll.

7.—The company sends yearly, at its expense, thirty-eight men, consisting of an engineer and a fireman from each division with the best fuel records, as its representatives to the Annual Convention of the International Railway Fuel Association.

As a result of the policy described and of many fuel-saving devices, the railroad moved 50% more gross ton-miles per pound of fuel in 1924 than in 1913.

V.—Transportation Department

About 20% of all locomotive fuel is consumed when the locomotive is not doing useful work.

(a) Avoidable losses include fuel used in firing up, unconsumed fuel drawn from fireboxes, from blowing off and cooling down at end of runs, and losses incurred while waiting for water, sand, and other supplies on roundhouse tracks. These can be very greatly lessened by reducing the frequency of these occasions, that is, by lengthening locomotive runs. It is not uncommon now to run freight locomotives over two or three freight divisions, and passenger locomotives over four before cooling them.   Tables 1 and 2 give some examples of modern practice.

TABLE 1.—PASSENGER RUNS OF 575 MILES AND MORE.

Road.	Limits of run.	Mileage.
Southern Pacific	Los Angeles, Calif., to El Paso, Tex	815
Missouri, Kansas and Texas	Parsons, Kans., to San Antonio. Tex	678
Union Pacific	Kansas City, Kans., to Denver, Colo	640
Southern Pacific	San Antonio to El Paso. Tex	620
Atchison, Topeka and Santa Fe.	Winslow. Ariz., to Los Angeles, Calif	602
Union Pacific	Denver, Colo., to Ogden, Utah	577
Southern Pacific	New Orleans, La., to San Antonio, Tex	575

Four locomotives only are used by the Southern Pacific to haul passenger trains from New Orleans to San Francisco, 2 490 miles.

TABLE 2.—FREIGHT RUNS OF 350 MILES AND MORE.

Road.	Limits of run.	Mileage.
Southern Pacific	El Paso to Del Rio, Tex	453
Southern Pacific	Sparks to Carlin, Nev	387
Southern Pacific	Houston, Tex., to Algiers, La	362
Southern Pacific	San Antonio to Fort Worth, Tex	352

(b) Good despatching will reduce the number of needless stops, and careful planning will avoid fuel waste from locomotives standing in sidings because of poor meeting points, and from idle time at terminals waiting for trains.

To stop a freight train of 66 cars weighing with its locomotive and tender 6 204 000 lb. and moving at a speed of 40 miles per hour, and to regain that speed requires a consumption of 380 lb. of coal.

(c) The economy of moving tonnage in as few cars and trains as possible and of reducing empty mileage is obvious.

TREND OF CURRENT AND FUTURE ECONOMIC DEVELOPMENT

Much idle time and fuel can be saved by carefully designed engine terminal facilities, such as hot-water boiler-washing plants, well located supply tracks, etc., and above all ample shops and modern tools to make repairs quickly. Additional shops and tools provided at El Paso on the Southern Pacific Lines have reduced the average time in shops of locomotives 25%, the effect of which is to add 33% to the number of locomotives assigned to that Division. On a division operated with Consolidation locomotives where water is exceptionally

good, 16 000 lb. of fuel are saved per locomotive per annum by the installation of a hot-water boiler-washing plant. Such improved facilities, while requiring large capital outlay, will increase the productive time of locomotives.

The burning of fuel in large terminals and shops should be centralized as far as practicable in order to eliminate machinery starting and stopping losses. Through free use of individual electric motors on large tools and groups of small tools the waste of running long lines of shafting and of operating tools needlessly when but one or two on the line are used, can be eliminated and substantial economies can be effected in the 16 000 000 tons of fuel consumed on railroads in shops, stations, etc. In a large railroad shop 20% of the power was saved by eliminating steam-pipe losses and 13% more by abandoning long lines of shafting and many belts through substituting electric for steam power. The changes represented a saving of $120 000 per year.

In locations where electrical energy can be purchased at lower cost than it can be produced by the railroad company, opportunities for substantial fuel savings are presented.

It is encouraging that the improvement in the work obtained from a pound of fuel is constant, but it is disappointing to reflect that European engineers are responsible for most of the improvements now in use on American locomotives. This is the case with the improved outside valve gear, superheaters and feed-water heaters, some details of which, however, have been designed by American engineers, but nevertheless it is mortifying to admit that the principles underlying these devices were not first used in the United States. The reason of course is that heretofore fuel has been so cheap and easily obtained that there was little inducement to pay much attention to its conservation; conditions have radically changed in the last few years, however.

The two principles affecting most profoundly the efficiency of stationary steam engines are compounding and condensing. Compounding alone may increase the efficiency of an engine 13%, and condensing alone may increase it as much as 30%, that is, condensing alone would increase the efficiency of fuel consumption of locomotives nearly as much as the brick arch, superheater, and feed-water heater combined, which as already shown amounts to 34.5 per cent. Compounding, which was used in the United States at one time much more than at present, will certainly come into use again; so that the combined effect of all the devices described, with compounding and condensing superadded, would be:

Brick arch when used has reduced fuel consumption to..   91.0%
The superheater when added has reduced it to..........   72.8%
The feed-water heater when added, to..................   65.5%
Using condensing engine may reduce it to.............   45.9%
Compounding may reduce it to........................   39.9%

European railroad engineers are fully alive to the advantages of using a condensing engine on the locomotive, as evidenced by descriptions that have appeared in the technical press in the past year or two.

Turbine Locomotives.—The Swiss Federal Railways* is testing a 10-wheel type locomotive, with a superheater, driven by a condensing steam turbine,

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* *Railway Review*, April 16, 1921.

located on locomotive frames in front of the smokebox. It is claimed that tests thus far show reduction in fuel consumption of 25% compared with compound locomotives of the usual construction, and very smooth running at high speeds on account of the reduction of heavy reciprocating parts.

The Swedish State Railway has also built a turbine locomotive.* It is claimed that this locomotive, known as a Ljungstrom locomotive, shows a fuel consumption of about 50% of that of a Pacific type locomotive, which it replaced.

The London and Northwestern Railroad has built an experimental locomotive driven by a condensing turbine which operates a generator, and the power is applied to the driving wheels through four electric motors of 275 h.p. each.

Electric Locomotives.—Where electric current can be generated from water power, trains can be moved very cheaply, but where the current is generated in a steam plant the advantages are much diminished. In eight large steam-electric plants studied in 1919 by the United States Geological Survey,† 12.2% of the energy in coal is converted into electric energy; transmitting this electric energy to the motors of an electric locomotive reduces the available power to 9.9% and the output is further reduced in the motors to 8.4% of the total original energy in the coal. The corresponding efficiency of a modern steam locomotive with arches, superheater, and feed-water heater is 8.1% under favorable conditions. While under existing conditions the advantage in favor of the electric locomotive may be more than that indicated, the spread is hardly enough to tempt capital to assume the expenses of interest, depreciation, tax, and maintenance that would have to be incurred in changing from steam power to electricity generated in a steam plant. The soundness of this conclusion was demonstrated by studies made of the Sierra Nevada and other heavy grades on the Southern Pacific, where trains are lifted as high as 6 854 ft. in 86 miles. Assuming the current to be generated in a steam station, the interest, taxes, depreciation on additional net plant required were found to be more than three times as great as the estimated savings to be obtained from electric operations.

Other considerations, such as smoke prevention and the increased capacity on heavy grades, may influence the problem, but the speaker recalls no instance of a change being made solely to save fuel where a steam-generated current was used. The steam locomotive is by no means as obsolescent as its critics contend. For example, a fuel consumption of 100 lb. per 1 000 gross ton-miles by electric locomotives is given‡ as applicable "to conditions universally obtaining on regular profiles." Data published by the Bureau of Railway Economics in September, 1921, show that the fuel consumption on 24 000 selected miles of steam railroad averaged but slightly more than 100 lb. per 1 000 ton-miles. Embraced in this mileage were the New York Central, Illinois Central, Chesapeake and Ohio and St. Louis Southwestern Railroads. On 3 142 miles, or 60% of Southern Pacific main-line mileage, the fuel con-

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* *Railway Review*, August 12, 1922.
† *Professional Paper No. 123*, U. S. Geological Survey.
‡ *Journal*. Am. Inst. Elec. Engrs., March, 1920.

sumption in the same month was approximately 100 lb., while on the Salt Lake Division, 543 miles, the fuel record in October, 1921, averaged only 91 lb. per 1 000 gross ton-miles, for all locomotives. During the entire month of October, 1924, sixty-two Southern Pacific freight engineers held records of having moved 1 000 gross ton-miles with a fuel consumption running between 48 and 59 lb. per 1 000 ton-miles, under "conditions universally obtaining on regular profiles," or but little more than one-half of what was considered a reasonable fuel consumption for electrically moved trains.

These are every-day performances; in the case of the Swedish experimental locomotive, and in records of electrically operated trains, however, the data relate either to experimental runs or to performances under exceptional conditions.

The performance of the modern steam locomotive in every-day service without favorable stage setting is shown in the records of some American roads. On one of them, 26 superheated Consolidation locomotives showed in daily service on a run with long grades of 21 ft. per mile consumptions of coal ranging from 55.8 to 71.7 lb. per 1 000 gross ton-miles, with an average of 63.5 lb. Still more creditable to the steam locomotives and the men who drive them are the records of individual runners. Engineer A in two trips with trains of 2 809 tons consumed 48.7 lb. per 1 000 gross ton-miles; Engineer P in eight trips with trains of 2 863 tons consumed 58.2 lb.; Engineer R in fourteen trips with trains of 2 411 tons consumed 58.3 lb.

Internal Combustion Engines.—These engines vary widely in thermal efficiency and weights per unit, as follows:

	Weight per brake horse-power, in pounds.	Percentage of brake thermal efficiency.
Automobiles	15	19.0
Diesel four-cycle	370	35.0
Diesel two-cycle	250	30.0
Airplanes	1.04	

The use of the automobile engine, except perhaps for short runs under exceptional conditions with light vehicles, is prohibitive on railroads, because although it is two and one-third times as efficient as the steam locomotive the present cost of its fuel is ten times as great.

The Diesel engine, however, shows the highest thermal efficiency of any known engine. As compared with the most modern locomotive, which transforms only about 8.1% of the heat energy of fuel into work, the Diesel engine transforms 35%, or more than four times as much. Its fuel (crude oil, commonly called Diesel oil) is not converted into gas as in gasoline engines, but is burned in the cylinder in the form of spray, and the expansive force of the products of combustion provides the motive power. Like the gas engine, however, it cannot start itself, but must use some form of variable speed transmission to start trains; the best way thus far devised consists of electric transmission, but the combined engine and generator plants have proven so heavy as to bar their use for heavy traction. Because of its high thermal efficiency, the Diesel engine offers opportunity to make great savings, and

its future is very hopeful. A locomotive of this type now being built for the Southern Pacific Lines by the Baldwin Locomotive Works seems to have overcome these handicaps and will soon be ready for trial.

The designs of an Italian Diesel locomotive show the following:

Weight in operation, approximately........	117 tons
Tractive effort at 18½ miles per hour......	19 840 lb.

The locomotive is started by highly compressed air stored in tanks and fed to cylinders at each end of the locomotive, which start it until sufficient speed is attained to start the Diesel engines, when the compressed air is cut off. The air tanks are recharged by a connection with the main engines. The estimated fuel saving over a steam locomotive is 66⅔ per cent.

This design shows progress in adapting the Diesel engines to locomotives of serviceable commercial sizes, particularly in solving the problem of weight, as the weight of a steam locomotive and tender to develop about 20 000 lb. of tractive effort is 150 tons.

The hope of the railroad executive for conserving fuel lies in:

1.—Substituting hydro-electric power for steam, as the best steam locomotives can now fully equal the efficiency of electric locomotives using current generated in steam power stations.

2.—Substituting compound condensing engines for the simple engines now used. From progress made in Europe, the solution of this problem appears encouraging.

3.—The discovery of a cheap, high-gravity fuel that can be used in some such engine as is used on automobiles and airplanes.

4.—Reduction of the weights of Diesel engines sufficiently to permit their use as locomotive engines and the development of a satisfactory variable speed transmission which will weigh much less than an electric generator and motor.

A Diesel electric locomotive built in Germany for the Russian Government shows an over-all efficiency between 21 and 27.4 per cent. A steam locomotive tested at the same laboratory two days later averaged a total efficiency from 7.6 to 8.67 per cent. The fuel consumption of the Diesel electric locomotive was about one-third that of the steam locomotive.

In a special report published in 1923, the National Industrial Conference Board appointed a committee to determine among other things:

1.—Do the industries in the United States need more or fewer engineers than the number now being graduated from engineering schools and colleges?

2.—What kind of men do the industries require from the engineering schools and colleges, and what should be the nature of their education?

The report states:

"The expansion of industry has been accompanied by vastly increased complexity of industrial operations, and the grouping of greater numbers of workers in corporate units has placed upon employers an increased social responsibility toward their employees and toward the life and affairs of the community. All of these changes have created problems of human, social and political relationships in which industry and society as a whole have become increasingly dependent upon trained technical and administrative leaders."

Solutions to these problems have been sought by railroad executives in the establishment of pensions for superannuated and incapacitated workers; in hospital departments, to furnish hospital service on the payment of nominal fees; by the establishment of group life insurance plans whereby employees, regardless of age and without physical examination, receive the benefits of life insurance at approximately one-third the cost of the current commercial rates; and by the establishment of rest and reading rooms at terminals where no opportunities exist for relaxation after working hours.

Continuing the report says: "Of the 41 600 000 persons who were gainfully employed in 1920, according to the United States Census for that year, less than 4% planned the activities and directed the energies of the whole working force." The same census figures show that since 1870 the number of administrators, supervisors, and technical experts has rapidly increased. In 1870, the number of persons engaged in administration and supervision was 170 000, or 1.25% of those gainfully occupied. In 1890, the corresponding data were 450 000, or 1.74%; in 1920, 1 510 000, or 3.6 per cent.

An examination of the personal records of the Chairmen and Presidents of the railroads of the United States shows that in 1922, of 32 Chairmen, 11, or 34%, had risen from the Engineering Department, and of 171 Presidents, 33, or 19%, had risen from the ranks of engineers. Based as they are on data taken from census reports, the conclusions of the National Industrial Conference Board show increasing demands for the services of trained men, and furnish the most reliable and valuable information obtainable on the matters under consideration.

As to schools, the report says:

"It is the business of the schools to train young men into fertile and exact thinkers, guided by common sense, who have a thorough knowledge of natural laws and of the means for utilizing natural forces for the advantage of men and the advancement of civilization. In other words, it is the business of engineering schools to produce, not finished engineers, but young men with a great capacity for becoming engineers, the goal being attained by the graduates only after years of development in the school of life."

Although a good technical expert may not develop into a good executive, nevertheless successful administrators should have certain characteristics, such as, a good grasp of the fundamentals of science; the ability to think logically and quantitatively; exactness of method and power of analysis; the habit of looking forward; sound economic theories; an unlimited capacity to learn; pronounced firmness combined with high sense of fairness and charity needed to control men; and the ability to write and speak clearly and correctly.

F. B. Jewett, Vice-President of the American Telephone and Telegraph Company and President of the Bell Telephone Laboratories, Incorporated, expresses these views:

"The industry which I am connected with is a highly technical one which is based on physics and chemistry and mathematics; but so are many of the big industries in our country and the more I see of them the more convinced do I become that the directors of these industries in the future are going to be drawn more and more from men who have a thorough grounding in the fundamentals on which the industries are based. In the old days it was not so, but each day we go on now, it becomes more clearly apparent that because

the industries are technical, because they are based on the fundamental science, they must be guided by men who have a thorough appreciation of the fundamentals because business is hardly more than mixing the dollar or cents with physics and chemistry and mathematics. Since engineering schools are the place where we expect to turn out men who are going to guide technical industries, we should turn out men who know the principles of the fundamental sciences; who know the method of combining fundamental sciences with the dollar and cents proposition and know English in a way to enable them to express clearly what they know, so that others may profit by it, because it does none of us very much good to be ever so wise if we cannot express our ideas to others.

"It seems to me that if the engineering schools of the future are going to perform their function for the industries they have got to get closer to the proposition of teaching and really inculcating the fundamentals. Let the matter of trying to make executives go, if you will. The men of executive ability will come to the fore in the general run of things, I think, provided the Lord gave them the right human traits and provided they have a thorough grounding in the fundamentals."

E. M. Herr, President of the Westinghouse Electric and Manufacturing Company, says:

"I have been very much impressed with the discussion and especially by the points brought out by the last speaker. I heartily endorse his view of the importance of the fundamentals and of the idea of patient plodding in the minds of men who are going into industry. There is no royal road, to my mind, to a place of commanding importance in industry, or in any other business, and I do not believe that it is the function of educational institutions to try to teach young men to become captains of industry and administrators, or occupy very great and important executive positions."

*   *   *   *   *   *   *   *

"While there is general agreement on this view that one of the leading essentials in education in relation to industry is an adequate training in and knowledge of fundamentals, there is wide variation in the meaning of the term 'fundamentals.' From the point of view of the industrialist, the term 'fundamentals' means a general groundwork of useful knowledge but, more especially, a training in the power to use this knowledge in effective thought and action.

"The fundamentals of engineering training may be stated as follows:

(a) Mathematics;
(b) The important principles of physics (mechanics, heat, light, sound, electricity, magnetism), chemistry, biology and geology, the interrelations of natural phenomena and the application of these principles to practical problems;
(c) The principles of economics and their application to industry and commerce;
(d) The principles that govern the relations between people, not only as applied to managers and men, but also as applied to governments and society;
(e) The history of nations;
(f) The art of clear and correct expression in speaking, writing, and drawing.

"The relative emphasis to be placed on each of these fundamentals is a matter to be determined for each engineering course."

In emphasizing the importance of training, one must studiously avoid withering the aspirations and stifling the ambition of the large numbers of men in railroad and industrial service who have not had the benefit of over-

much school or any college preparation. These men outnumber vastly the high school and college men, and to their credit be it said that the 65% of Chairmen and 81% of Presidents remaining after taking account of those who have risen from the ranks of engineers have been drawn largely from the men who have had none of the benefits of higher education. Without mentioning them, the names of a number of the ablest and most distinguished railroad Chairmen and Presidents in the United States, who have by sheer ability and pluck risen from lowly ranks and have actually built their own careers and builded admirably well, suggest themselves.

Again quoting from the report to the National Industrial Conference Board:

"Sympathetic guidance of both the college man and the non-college man presupposes a careful evaluation of the work of all employees and assurance to them that talent and effective work will be known to and appreciated by the management. Educators report a growing feeling among undergraduates that in most large corporations a young employee often is lost in a department and for many months may not come under the observation of the leading men of the organization. A large public utility company, the two largest manufacturers of electrical apparatus, and a few other large corporations which have unusual and enlightened personnel departments, in many cases are getting the pick of the young talent, because of their attention to this problem."

"These are some of the ways in which industry can supplement the work of the educators," and profiting by their example thus can the present generation help those who are to "carry on" when it shall have passed.

THE REINFORCED CONCRETE ARCH IN SEWER CONSTRUCTION: A REVIEW OF PAST PRACTICE IN DESIGN AND AN ACCOUNT OF RECENT STUDIES IN ST. LOUIS, MISSOURI

By Charles E. Sharp,* Jr., Esq.

To Be Presented September 2, 1925.

Synopsis

In this paper, the elastic design of the reinforced concrete sewer arch is described. Conditions of loading common to such sewers are discussed in detail and formulas are presented for corresponding vertical and horizontal loading, due to the back-fill of plastic soil. Conditions of arch support are also analyzed, leading to the conclusion that complete restraint is rarely accomplished in sewer arches and that a certain amount of hinged action must be recognized. Fundamental equations for the determination of arch stresses are presented and discussed and, for arches of a particular shape, equations for moments and stresses are deduced in which the variables are the radius of the arch and the depth of earth-fill.

Introduction

The position of the reinforced concrete sewer arch in engineering practice has always been somewhat confused. The principles involved in the design of such a structure have not been studied and developed to the same extent as for bridge construction. As St. Louis, Mo., was one of the pioneers in the use of this type of sewer structure, an account of the St. Louis practice will be, for practical purposes, a statement of the development from the beginning.

The reinforced concrete arch sewer came into use about 1900. Until that time large sewers in the United States were gravity arches of massive masonry construction. The appearance and bulk of these sewers had become firmly fixed in the minds of municipal engineers. When the possibilities of economy through the use of reinforced concrete for sewer purposes became evident, concrete designers were called in to co-operate with sewer engineers. These designers were accustomed to working with fairly high concrete stresses and with the relatively thin slabs common in building construction. Their proposed designs were so slender and apparently fragile (from the viewpoint of the men accustomed to massive masonry construction), that they were never quite acceptable. As a compromise, safety factors without end were crowded

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into the design, and arbitrary additions to the concrete thickness, often without any reduction in the reinforcement, were quite common. Even after these increases to the mass of the structure were made, the results were widely criticized, and predictions were rife that such construction could not be counted on for long life, particularly as the reinforcement would rapidly disintegrate when the concrete was covered with earth and was constantly damp.

Gradually, this attitude has been modified. Within recent years engineers have come to feel they can calculate, with reasonable accuracy, the possible stresses in this type of structure and can construct with safety to the dimensions indicated by these calculations. This change in viewpoint has been due in part to the confidence resulting from their greater familiarity with the material and in part to the fact that they had almost forgotten the old massive types. It has been forced by the increase in sizes of sewers and the necessity of stricter economy in materials.

The 32-ft. sewers required for the River des Peres Drainage Works in St. Louis involve construction costs mounting into millions. In designing these sewers, it became evident that a re-study of the reinforced concrete sewer arch from every aspect was essential. These studies were made by the writer in consultation with the Engineers of the Division of Sewers and Paving of the City of St. Louis. They involve an analysis of the three principal factors entering into design, that is, the possible earth pressures acting on the arch, the conditions under which the arch is supported, and the bending moments and stresses resulting from various combinations of loads and bearings.

EARTH PRESSURES

It has been customary to design sewer arches for certain dead loads resulting from the back-fill over the structure. It is true that sometimes special designs are required to support surface structures, and that occasionally moving loads and even engine loads must be considered. These special sections, however, are unusual and rarely affect as much as 5% of the whole sewer.

In the earlier designs it was customary to assume the sewer arch as constructed in a trench with vertical sides and subject to the weight of earth acting vertically. It was understood that certain horizontal pressures must exist, but because of the inability of designers to determine accurately their intensities, and of the fact that such horizontal pressures as existed would reduce the bending moments resulting from vertical loads, these pressures were ordinarily ignored.

In later work arbitrary values were assigned to the intensity of horizontal pressure, but there was no general agreement in this respect. In St. Louis, it was at one time customary in case of plastic soils to assume the horizontal pressures at any point. Again, it was recognized that in plastic soils compression in the vertical direction could not exist without co-ordinate horizontal pressure. It was also clear that the supposed vertical columns of earth could not rest on the curved surfaces of an arch unless certain horizontal forces prevented them from sliding off. On this theory a formula was devel-

oped, that was expected to indicate a minimum horizontal pressure which must occur. Although it was known that greater pressures probably did occur, designers did not feel safe in considering them. In recent studies advantage has been taken of the work of the Society's Special Committee on the Bearing Value of Soils for Foundations and of the research work done at several universities on earth pressures, to develop somewhat more definite values for this phase of loading.

Support of the Arch Ring

Formerly the arch was assumed not merely to be supported at the invert, but to be fixed in direction at this point. It was further assumed, where arches were built on rock, that the contact with the rock was sufficient to produce this fixture. Where the sewer was built in plastic soils, it was customary to assume the complete structure as an elastic ring with the bearing distributed uniformly across the lower half. In this instance the sewer, symmetrically loaded, was considered to be fixed in direction at the crown and at the center of the invert. It is now found advisable to analyze stresses not only with the abutments completely restrained, but also on the assumption that the arch is free to rock at the abutment bearings. In studying the change of direction of a hinged arch at the theoretical hinges, the angular movement required for full loading was found to be extremely small. Further studies indicated that even where the sewer was built on rock it would not be unreasonable to expect deflections that would permit the apparent rocking or hinge action of the arches. It became clear, therefore, that the conditions, heretofore supposed, of full restraint of the ends of the arch, could only be secured when the arch was built in rock cuts of considerable depth and abutted against solid walls for several feet above the invert. It is also clear that under this latter condition the arch may be designed as fixed in direction and the span considered only as between the points of abutment against the upper rock surface.

A review of the research work on plastic soils also indicated that the vertical weight of the structure need not be distributed over its full width and that sufficient bearing power in hard clays, shales, and similar soils, can be found by utilizing a comparatively small area under each abutment. This study makes it clear that, except in unusually soft ground, the heavy inverts heretofore used are unnecessary.

Both these factors in design will be described more fully hereafter; the third factor, that is, the determination of stresses due to various combinations of load and support, is analyzed in detail.

Structural Design of the River des Peres Sewer
General

The River des Peres Sewer in St. Louis will be composed of two distinct sections each approximately 2 miles in length, the upper one being of single semi-circular form, 32 ft. in diameter, and the lower of double semi-circular form, with barrels 29 ft. in diameter. The proposed earth-fills over

the crown vary in both sections from about 3 ft. to about 20 ft. Boring tests showed that the 4 miles of sewer will rest successively on fire clay, shale, and limestone.

Conditions of Support

In the first studies of typical sections, current practice was followed, taking for granted that when a sewer arch abuts on rock, its ends must be considered as fully fixed or restrained; that when it abuts on compressible soil, the arch should not be considered as stopping at its base, but should turn inward and, with the invert, form a wholly continuous ring, as shown in Figs. 1 and 2.

The greatly extended length of the arch ring in the latter case gave uniformly large stresses; and in the former, large bending moments were found at the abutment by combining the earth pressure with the water pressure that would be exerted while the sewer was flowing at its designed capacity.

HALF SECTION IN ROCK   FIG. 1.
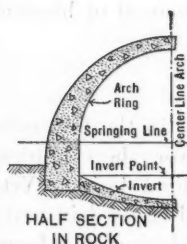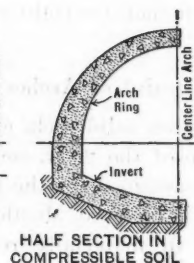
HALF SECTION IN COMPRESSIBLE SOIL   FIG. 2.
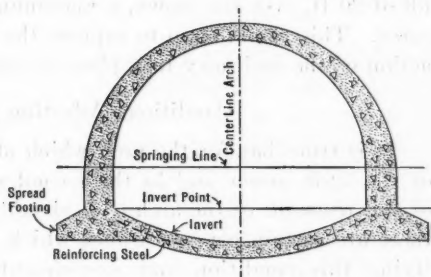
FIG. 3.

It seemed that an arch based on such assumptions could bear some profitable investigation before being accepted as the nearest approximation to the truly economical design, and that by confining the arch support (even in compressible soil) to a small area at the abutment, arch stresses for both rock and clay foundations would become more nearly comparable, and also more nearly equal. This confinement was effected by using a spread footing at the abutment, the spread varying with the vertical load of earth-fill and the carrying capacity of the soil. The footing itself was considered a part of the arch in the subsequent design, as shown in Fig. 3.

It was assumed that the soil-bearing value of $2\frac{1}{2}$ tons per sq. ft. for the softest soil was low enough to prevent any lateral flow of the loaded soil.

In order to design by the usual arch formulas, it is necessary that the span length between supports be invariable, that is, that the abutments should admit no appreciable lateral movement. To insure this condition for the sewer in earth, the flat invert was made thick enough to resist any inward movement of the abutments. By tying it to the spread footing with reinforcing bars placed along its center line (Fig. 3), it would also resist any outward movement without becoming an integral part of the main arch ring or assuming an appreciable part in distributing the load to the soil. In rock, this condition was insured by the bond between the jagged surface and the abutting arch ring. It is almost inconceivable that, under ordinary loading

of the sewer arch, any outward movement would take place; in any event, however, these bars will assist in preventing temperature cracks.

Despite this apparent rigidity of the supports, the spread footing suggested possible hinge action around its own center point. A clockwise rotation of the left-hand abutment would incline its center line outward at the bottom. The spread footing, being integral with the arch would dig into the soil on the invert side, and rise on the outer side. The conditions might also be just reversed.

A theoretical value for this sinkage of the extreme edge of the footing may be found by computing the angular change $\left(\int \dfrac{M\,d\,s}{E\,I} \right)$, in the direction of the arch ring, and, consequently, of the bottom of the footing, and multiplying this by the distance from the center line out to the edge of the footing. For the hinged arch, with soil bearing at $2\frac{1}{2}$ tons per sq. ft. and with a maximum fill of 20 ft. over the crown, a maximum vertical movement of $\frac{1}{8}$ in. was indicated. This would seem to support the idea that a certain amount of hinging action of the arch may take place in clay.

Conditions Affecting Restraint of Arches

It is true that for the arch which abuts on solid rock, as in the first mile of the 32-ft. sewer and in the second mile of the 29-ft. sewer, the condition of full restraint of the arch ends should be assumed for the final design. Yet, there are certain considerations, which, it is believed, should be used in modifying this condition and, consequently, in materially reducing the large bending moments found by the rigid analysis.

In the first place, it was found in the analysis of a typical arch for various kinds of loading, amounts of fill, and conditions of restraint, that all maximum abutment moments were positive* in sign, that is, produced compression in the extrados of the arch and tension in the intrados. An earth slide against the side of the arch during construction would give large negative moments at the abutment, but this danger was neglected for design purposes for the following reasons:

1.—Only a certain longitudinal section of the sewer could be affected at one time; the adjacent sections would tend to support it.

2.—Failure could occur only during the time of construction, when no back-filling had been done and when repairs could be made more cheaply than provision against such failure along the entire line of the sewer.

In the second place, two factors peculiar to construction in rock would tend to reduce the possible deflection of the footing to such an extremely small value that even some hinging action is conceivable in rock. These factors are (1) as the supporting ground becomes harder, the necessary width of footing decreases; and (2), the value, $\int \dfrac{M\,d\,s}{E\,I}$, approaches zero for com-

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* In order to realize these positive moments, the arch ends would have either to develop the tension on the intrados side or to be encased in a wall of sufficient length and roughness to render the concrete and rock an integral mass, thus reducing the tension in the intrados to a negligible quantity. By placing a joint along each side of the thin invert, the possibility of developing tension was largely eliminated.

plete fixation. The deflection of the footing edge, being the product of these small quantities, would be almost insignificant.

In the third place, the analysis showed that the moments at the crown and the springing line for the hinged condition of the arch ends were larger than those for the fixed condition. Also, the springing line moments for the hinged condition were about equal to or greater than the abutment moments for the fixed condition. Now, since in practical construction it is undesirable to decrease the size of the arch ring below the springing line, these hinged-arch values at the springing line may be used with safety to design the abutment also.

Finally, where the arch was found to rest against a rock wall of height sufficient to realize full fixation, it was analyzed as a shortened arch ring, the resulting moments being smaller than for the full length of ring.

Conditions of Loading

1.—*Earth.*—The full weight of superimposed earth together with that of the concrete arch ring was assumed as the vertical loading in all cases. Experiments* show that unless the back-filled part of the trench in which a sewer arch is to be built, has a depth greater than its width, no appreciable reduction in the vertical loading due to arch action of the soil is permissible. In the River Des Peres Arch, the maximum back-fill was less than the width of the trench.

In designing the arch, 2-ft. widths for the vertical columns of earth (including the part of the arch ring immediately below), were taken as sufficiently accurate. The weight of earth was assumed to be 100 lb. per cu. ft. It happens that where the fill is slight no appreciable live load will occur, and where the fill is heavy, a street will be built over the sewer, likewise with negligible live loads. In the isolated cases, in which the sewer passes beneath railroad tracks, an additional earth-fill of 10 ft. will represent the live loads.

Many theories have been advanced concerning the horizontal pressure that the back-fill is able to exert on the sides of a sloping wall (in this case the arch ring). A modification of the ones developed by Jacob Feld, Assoc. M. Am. Soc. C. E., and supported by his series of experiments,† was used in this design. In it, horizontal pressures were considered to act against the side of the arch. For convenience, they were taken in horizontal zones of equal width. The pressure for each zone was found from the formula, $H = W.h_1.w.k$, in which, W = the unit weight of earth; h_1 = the distance from the finished ground surface to the center line of the zone; w = the width of the zone; and, k = the quantity found from Equations (a) and (b).

Case 1.—Slope of Arch Extrados Less Than 60°.—

$$k = \left(\frac{\cos (\phi - \alpha)}{(r + 1) \cos \alpha} \right)^2 \dots\dots\dots\dots\dots\dots\dots (a)$$

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* *Bulletin 31,* Eng. Experiment Station, Iowa State Coll. of Agriculture.
† *Buildings,* Monthly Issue of *Engineering and Contracting,* May 23, 1923, pp. 1163-1165.

in which,

$$r = \sqrt{\frac{\sin(\phi + \phi') \sin(\phi - E)}{\cos(\phi + \alpha) \cos(\alpha - E)}};$$

ϕ and $\phi' = 30°$ each;

E = slope of ground = $0°$; and

$\alpha = 90°$ — slope of arch extrados.

Case 2.—Slope of Arch Extrados Greater Than 60°.—

$$k = \frac{1}{3} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots(b)$$

Now, in general, horizontal earth pressures reduce, throughout the arch, the stresses which would be developed by purely vertical loading. It was necessary, therefore, in allowing the possibility of some horizontal pressure, to note cases where full horizontal loading to the bottom of the arch could not be exerted. The position of the rock surface determines the point below which it could not exist. Three such positions were taken as critical, that is, rock at the springing line, rock at the arch end, and rock considerably below the arch end. The existence of rock above the springing line was, of course, disregarded because the earth back-fill will extend at least as far down as the springing line, and exert horizontal pressures. Where the rock surface was at the springing line or abutment (in the analysis of a typical arch), the arch was considered as fixed, and where considerably below the springing line, as hinged.

Table 1 gives the possible combinations of horizontal loading and restraint where the two abutments of the arch are not necessarily supported in the same way. Additional cases of pressure of a sliding mass of earth during construction are included. The value of this pressure was arbitrarily taken as equal to the sum of the ordinary horizontal loads between the springing line and the abutment, and double for impact. Only the single barrel arch was analyzed for these special cases of loading; as they produced no important maximum moments in this arch, they were neglected in the analysis of the double barrel arch.

TABLE 1.—SIMULTANEOUS CONDITIONS OF ARCH LOADING AND RESTRAINT.

I.—Arch fixed at both ends.......	(A)	Horizontal loads to abutment on both sides.
	(B)	Horizontal loads to springing line on both sides.
	(C)	Sliding earth loads on left side only.
	(D)	Horizontal loads to abutment on right side and to springing line on left side.
II.—Arch hinged at both ends....	(E)	Horizontal loads to abutment on both sides.
	(F)	Horizontal loads to springing line on both sides.
	(G)	Sliding earth loads on left side only.
III.—Arch fixed at right end, hinged at left..............	(H)	Horizontal loads to springing line on both sides.
	(I)	Sliding earth loads on left side only.
IV.—Arch hinged at right end, fixed at left...............	(J)	Horizontal loads to abutment on both sides.
	(K)	Sliding earth loads on left side only.
	(L)	Horizontal loads to abutment on right side, and to springing line on left side.
V.—Shortened arch* fixed at both ends.....................	(M)	Horizontal loads to top of rock surface.

* Rock surface at the springing line.

2.—Water.—The dimensions of the River Des Peres Sewer were made such that it would carry off all storm water with a head varying from zero to 5 ft. above the crown intrados. In the typical design, the arch was analyzed for three lower water levels, but all these gave lesser stresses than those resulting from a full water level. This is because the water pressure, increasing along the arch intrados down to the invert, tends to nullify the relieving effect of the horizontal earth pressure. In other words, it adds to the stresses caused by vertical earth loading.

3.—Temperature.—A drop of 50° Fahr. in the temperature of the material used in the arch construction was thought to be sufficient. This would accommodate a change from about 100° during construction to about 50° after back-fill. An equal rise in temperature was also used, accommodating a change from 30° Fahr. (concreting in freezing weather) to about 80 degrees.

4.—Rib-Shortening.—In the fixed arch the maximum rib-shortening was found to be only 10% of the shortening due to temperature, and, therefore, this increment was added to the temperature computation throughout.

Formulas and Equations of Computation

In all cases, a symmetrical arch was chosen for analysis, and the assumed voussoir joints were located so as to maintain $\dfrac{d\,s}{I}$ constant. The slight variation in arch thickness as finally developed will admit of a large amount of unsymmetry, that is, points of maximum moment can shift considerably around the arch ring (due to an unsymmetrical arch) and still be met with a section of sufficient resistance.

The ordinary formulas for thrust, shear, and moment at the crown, and for moment at any point were used in the case of the symmetrical fixed arch. Formulas were developed for these forces in the case of the hinged arch, and for the arch fixed at one end and hinged at the other. For the double arch, the methods of Morley* were used in deriving the formulas. Morley analyzes the hinged arch using the hinged end as origin. He also deals with the moment at the end as a function of the moment of the bending moment diagram about the left support. This reasoning can be modified to apply to the double arch.

Derivation of Arch Formulas

For convenience, all the terms used in these derivations are collected and here defined. Moments are considered positive when to the right of a given section they produce clockwise rotations. Shears are considered positive when to the left of a given section they produce downward pulls. The term "inner springing line" means the springing line at the top of and to either side of the center wall of a double arch. Unless otherwise noted, all summations, Σx, Σy, ΣM, etc., for the single arch extend from one abutment to the other. For the double arch, unless otherwise noted, they extend from one abutment to the inner springing line. All single arches are symmetrical

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* "Strength of Materials", pp. 201–203, and 348–361.

about the crown. All double arches are symmetrical about the center wall. The loading may be unsymmetrical in all cases of the single or double arch. The dimensions, forces, constants, etc., may be expressed in any consistent system of units.

Definitions

The definitions are as follows (Fig. 4).

LEFT HALF
SHOWING VARIOUS LOADS

RIGHT HALF
SHOWING LINEAR QUANTITIES
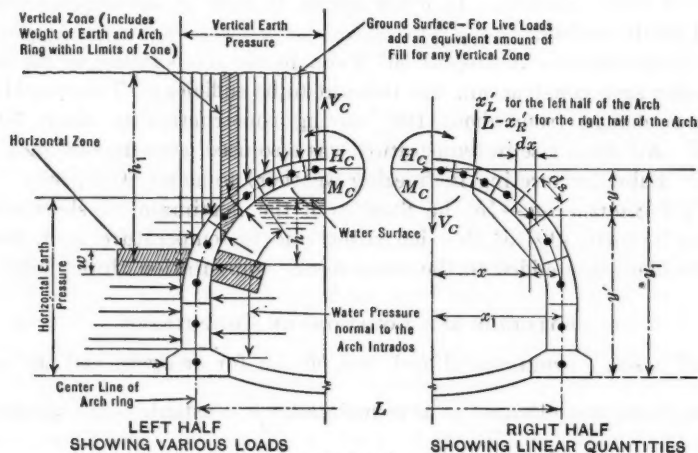
FIG. 4.

Forces.—

M_A = Actual bending moment at the left abutment.

M_B = Actual bending moment at the right abutment.

M_C = Actual bending moment at the crown.

M_D = Actual bending moment at the central abutment of a double arch.

M_K = Actual bending moment at the inner springing line.

M_L = Actual bending moment at a given voussoir point in the arch ring to the left of the crown.

M_R = Actual bending moment at a given voussoir point in the arch ring to the right of the crown.

M_1 = Bending moment at any point due to treating a partly fixed arch as fully hinged at both abutments.

m_A = Moment of all external forces for a given type of loading occurring between the crown and the left abutment, taken around that abutment.

m_B = Opposite hand of m_A.

m_L = Moment of all external forces for a given type of loading occurring between the crown and a given voussoir point to the left of the crown, taken around that point.

m_R = Opposite hand of m_L.

μ_L = Moment of all external forces for a given type of loading occurring between the left abutment and a given voussoir point to the left of the double arch center wall, taken around that point.

μ_K = Same as μ_L, except for the inner springing line to the right or left of the center wall.

$\mu_R =$ Opposite hand of μ_L.

$H_A =$ Horizontal reaction at left abutment.

$H_B =$ Horizontal reaction at right abutment.

$H_C =$ Horizontal reaction or thrust at the crown.

$V_A =$ Vertical reaction at left abutment.

$V_B =$ Vertical reaction at right abutment.

$V_C =$ Vertical reaction or shear at the crown.

Linear Quantities.—

$X =$ Horizontal distance from crown to center line of a given voussoir.

$X_L' =$ Horizontal distance from the left abutment to the center line of a voussoir to the left of the crown.

$X_R' =$ Horizontal distance from the left abutment to the center line of a voussoir to the right of the crown.

$X_1 =$ Horizontal distance from the left or the right abutment to the crown ($X_1 = X + X_L'$).

$X_2 =$ Horizontal distance from the crown to the inner springing line.

$X_G =$ Horizontal distance from the left reaction of a beam to the center of gravity of its bending moment diagram.

$ds =$ Length of a voussoir measured along the center line of the arch ring.

$dx =$ Horizontal projection of ds.

$L =$ Length, center to center, between abutments or, in the case of a double arch, between one abutment and the center wall.

$L_1 =$ Length, center to center, between one abutment and the inner springing line.

$y =$ Vertical distance from the crown to the center line of a given voussoir.

$y' =$ Vertical distance from abutment to the center line of a given voussoir.

$y_1 =$ Vertical distance from crown to abutment.

$y_2 =$ Vertical distance from crown to inner springing line.

Miscellaneous.—

$A =$ Area of the bending moment diagram of a beam.

$a =$ Expansion coefficient of arch material per degree Fahrenheit.

$C =$ Average unit compression, in pounds per square inch, occurring throughout the arch ring.

$E =$ Modulus of elasticity for the arch material.

$I =$ Moment of inertia of arch section about a tangential axis through the center line of the arch at any given voussoir point.

$i_A =$ Change in the tangent of the slope of the beam or arch of the left abutment.

$i_D =$ Change in the tangent of the slope of the beam or arch at the center wall.

$i_K =$ Change in the tangent of the slope of the beam or arch at the inner springing line.

$t =$ Temperature change, in degrees Fahrenheit.

$N =$ Number of voussoirs in the arch ring.

It should be noted that when any voussoir, such as the spread footing portion of the arch ring, is much thicker than a number of other voussoirs having more or less equal thickness, no attempt to make its $\dfrac{d\,s}{I}$ value equal to that of the other voussoirs need be made. If, for instance, its $\dfrac{d\,s}{I}$ value were one-tenth of a convenient $\dfrac{d\,s}{I}$ for the other voussoirs, its X, my, mx, etc., may be divided by 10 in compiling the mechanical summations, $\Sigma\,x$, $\Sigma\,my$, $\Sigma\,mx$, etc. Also, in computing N, the spread footing may be counted as only one-tenth of a regular voussoir.

Single Arch Fixed Both Ends

Earth and Water Pressure.—From Fig. 4 :

$$M_L = M_C + H_C\,y + V_C\,x \pm m_L \dots\dots\dots\dots\dots\dots (1)$$

and,

$$M_R = M_C + H_C\,y - V_C\,x \pm m_R \dots\dots\dots\dots\dots (2)$$

Now, in order to determine M_L and M_R for various points, it is necessary to solve for M_C, H_C, and V_C. The three equations for this solution are based on the facts that, for an arch completely restrained at both abutments, the total rotation, $\Sigma\,M\,\dfrac{d\,s}{E\,I}$, the total change in length of span, $\Sigma\,M\,y\,\dfrac{d\,s}{E\,I}$, and the total change in level between the arch ends, $\Sigma\,M\,x\,\dfrac{d\,s}{E\,I}$, are each equal to zero. In all cases E is a constant, while $\dfrac{d\,s}{I}$ has been made constant by taking appropriate lengths for the voussoirs along the arch ring.

Thus,

$$\Sigma\,M,\ \Sigma\,M\,y,\ \Sigma\,M\,x = 0$$

or, substituting the equal of M from Equations (1) and (2) :

$$N\,M_C + H_C\,\Sigma\,y \pm \Sigma\,m = 0 \dots\dots\dots\dots\dots (3)$$

$$M_C\,\Sigma\,y + H_C\,\Sigma\,y^2 \pm \Sigma\,m\,y = 0 \dots\dots\dots\dots (4)$$

$$V_C\,\Sigma\,x^2 \pm \Sigma\,(m_L - m_R)\,x = 0$$

or,

$$V_C = \mp\,\frac{\Sigma\,(m_L - m_R)\,x}{\Sigma\,x^2} \dots\dots\dots\dots\dots\dots (5)$$

By solving Equations (3) and (4) simultaneously,

$$H_C = \mp\,\frac{N\,\Sigma\,m\,y - \Sigma\,m\,\Sigma\,y}{N\,\Sigma\,y^2 - (\Sigma\,y)^2} \dots\dots\dots\dots (6)$$

$$M_C = \frac{\mp\,\Sigma\,m - H_C\,\Sigma\,y}{N} \dots\dots\dots\dots (7)$$

The lower signs are to be used for earth pressure; the upper ones for water pressure.

Temperature.—Similarly to Equations (1) and (2), for temperature stress:

$$M_L = M_R = M_C + H_C \, y \dots \dots \dots \dots \dots (8)$$

Since here $V_C = 0$, only two equations are required to solve for M_C and H_C. These are furnished by the facts that $\Sigma \, M = 0$ (as for earth and water pressure) and $\Sigma \, M \, y \, \dfrac{d \, s}{E \, I} \, \pm \, a \, t \, L = 0$, in which $a \, t \, L$ is the amount the span would increase if the abutments were free to move, and $\Sigma \, M \, y \, \dfrac{d \, s}{E \, I}$ is the amount the span is prevented from moving by the fixation of the abutments.   Hence,

$$M_C \, \Sigma \, y + H_C \, \Sigma \, y^2 = \mp \, \frac{I \, E \, a \, t \, L}{d \, s} \dots \dots \dots \dots \dots (9)$$

and,

$$N \, M_C + H_C \, \Sigma \, y = 0, \ \text{or} \ M_C = - \, \frac{H_C \, \Sigma \, y}{N} \dots \dots \dots \dots (10)$$

Substituting this value of M_C in Equation (9), and solving for H_C,

$$H_C = \mp \, \frac{I \, E \, a \, t \, L}{d \, s \left(\Sigma \, y^2 - \dfrac{(\Sigma \, y)^2}{N} \right)} \dots \dots \dots \dots \dots (11)$$

Note that the lower sign is to be used for temperature decrease; the upper sign for temperature increase.

Rib-Shortening.—In the formulas of temperature stress for H_C and M_C, the product $E \, a \, t$ represents the average unit compression in the arch ring due to a temperature decrease of t degrees.   For rib-shortening, by simply replacing this product with C,

$$H_C = \frac{I \, C \, L}{d \, s \, \Sigma \, y^2 - \dfrac{(\Sigma \, y)^2}{N}} \dots \dots \dots \dots \dots (12)$$

$$M_C = - \, \frac{H_C \, \Sigma \, y}{N} \dots \dots \dots \dots \dots (13)$$

Single Arch Hinged Both Ends

Earth and Water Pressure.—As for the fixed arch,

$$M_L = M_C + H_C \, y + V_C \, x \pm m_L \dots \dots \dots \dots \dots (1)$$

and,

$$M_R = M_C + H_C \, y - V_C \, x \pm m_R \dots \dots \dots \dots \dots (2)$$

Since for the hinged condition the bending moments at the hinges or abutments equal zero,

$$M_A = M_C + H_C \, y_1 + V_C \, x_1 \pm m_A = 0 \dots \dots \dots \dots (14)$$

$$M_B = M_C + H_C \, y_1 - V_C \, x_1 \pm m_B = 0 \dots \dots \dots \dots (15)$$

In order to solve for M_C, H_C, and V_C, one equation in addition to Equations (14) and (15) is needed. For the hinged arch, where the total rotation, $\Sigma M \dfrac{d\,s}{E\,I}$, does not equal zero, it is necessary to use $\Sigma M y' \dfrac{d\,s}{E\,I}$ instead of $\Sigma M y$ $\dfrac{d\,s}{E\,I}$ for expressing the total change in span length. The reason for this is that $\Sigma M y$ depends for its zero value on a simultaneous zero value of ΣM, while $\Sigma M y'$ $\left(\text{multiplied by the constant } \dfrac{d\,s}{E\,I}\right)$ represents the sum of all span changes measured along the base line of the arch, and is, therefore, independent of the total rotation change, $\Sigma M \left(\dfrac{d\,s}{E\,I}\right)$. The additional equation is, therefore, $\Sigma M y' = 0$, from which,

$$M_C \,\Sigma\, y' + H_C \,\Sigma\, y\, y' \pm \Sigma\, m\, y' = 0 \dots\dots\dots\dots\dots(16)$$

Solving Equations (14), (15), and (16), simultaneously,

$$H_C = \mp \frac{\Sigma\, m\, y' - \dfrac{m_A + m_B}{2}\, \Sigma\, y'}{\Sigma\, y\, y' - y_1\, \Sigma\, y} \dots\dots\dots\dots(17)$$

$$M_C = \mp \frac{\Sigma\, m\, y' - H_C\, \Sigma\, y\, y'}{\Sigma\, y'} \dots\dots\dots\dots(18)$$

$$V_C = \mp \frac{m_A - m_B}{L} \dots\dots\dots\dots\dots(19)$$

Note that the lower sign is used for earth pressure; the upper one for water pressure.

Temperature.—As for the fixed arch,

$$M_L = M_R = M_C + H_C\, y \dots\dots\dots\dots\dots(8)$$

In this case, the total change in length of span is:

$$\Sigma M y' \frac{d\,s}{E\,I} \pm a\, t\, L = 0$$

or,

$$\Sigma M y' = \mp \frac{I\, E\, a\, t\, L}{d\,s} \dots\dots\dots\dots\dots(20)$$

Since for the hinged condition, $M_A = 0$,

$$M_A = M_C + H_C\, y_1, \text{ or } M_C = -\, H_C\, y_1 \dots\dots\dots\dots(21)$$

Substituting the value of M_C from Equation (21) in Equation (8),

$$M_L = M_R = -\, H_C\, (y_1 - y) \dots\dots\dots\dots(22)$$

From Fig. 6,

$$y_1 - y = y'$$
$$M_L = M_R = -\, H_C\, y'$$
$$\Sigma M y' = -\, H_C\, (y')^2 = \mp \frac{I\, E\, a\, t\, L}{d\,s}$$

or,

$$H_C = \pm \frac{I\,E\,a\,t\,L}{d\,s} \dots\dots\dots\dots\dots\dots\dots\dots\dots(23)$$

The lower sign applies for temperature decrease, the upper one for increase.
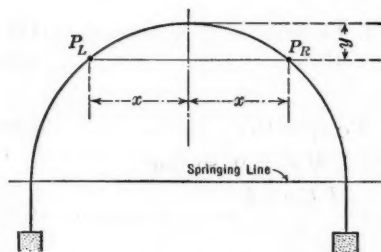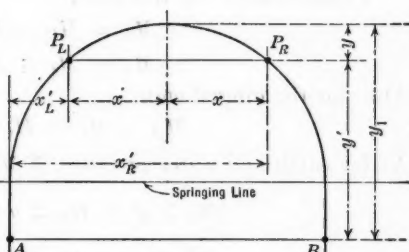
FIG. 5.   FIG. 6.

Rib-Shortening.—As before, replace $E\,a\,t$ in the temperature formula with C, so that,

$$H_C = \frac{I\,C\,L}{d\,s} \dots\dots\dots\dots\dots\dots\dots\dots\dots(24)$$

$$M_C = -\,H_C\,y_1 \dots\dots\dots\dots\dots\dots\dots\dots(21)$$

SINGLE ARCH HINGED AT LEFT END, FIXED AT RIGHT END

Earth and Water Pressure.—As for the fixed arch,

$$M_L = M_C + H_C\,y + V_C\,x \pm m_L \dots\dots\dots\dots\dots\dots(1)$$

and,

$$M_R = M_C + H_C\,y + V_C\,x \pm m_R \dots\dots\dots\dots\dots\dots(2)$$

Also, for the hinged end,

$$M_A = M_C + H_C\,y_1 + V_C\,x_1 \pm m_A = 0 \dots\dots\dots\dots\dots(14)$$

In order to solve for M_C, H_C, and V_C, two equations in addition to Equation (14) are required. As for the arch hinged at both ends, $\Sigma\,M\,y' = 0$. Similarly, $\Sigma\,M\,x' = 0$, so that,

$$M_C\,\Sigma\,y' + H_C\,\Sigma\,y\,y' \pm \Sigma\,m\,y' = 0 \dots\dots\dots\dots\dots\dots(16)$$

$$M_C\,\Sigma\,x' + H_C\,\Sigma\,y\,x' + V_C\,(\Sigma\,x\,x_L' - \Sigma\,x\,x_R') \pm \Sigma\,m\,x' = 0 \dots\dots(17)$$

However,

$$\Sigma\,x\,x_L' - \Sigma\,x\,x_R' = \Sigma\,x\,(x_L' - x_R') \text{ for half the arch}$$

or (see Fig. 7):

$$= -\,\Sigma\,x\,(2\,x) = -\,\Sigma\,x^2 \text{ for the whole arch.}$$

Substituting this value of $\Sigma\,x\,x_L' - \Sigma\,x\,x_R'$ in Equation (17), and solving Equations (14), (16), and (17), simultaneously,

$$H_C = \frac{\mp\,\dfrac{\Sigma\,m\,y'}{\Sigma\,y'}\left(\dfrac{\Sigma\,x^2}{x_1} + \Sigma\,x'\right) - \dfrac{m_A\,\Sigma\,x^2}{x_1} \pm \Sigma\,m\,x'}{\dfrac{\Sigma\,y\,y'\,\Sigma\,x'}{\Sigma\,y'} - \Sigma\,y\,x' + \dfrac{\Sigma\,x^2\,\Sigma\,y\,y'}{x_1\,\Sigma\,y'} - y_1\,\dfrac{\Sigma\,x^2}{x_1}} \dots\dots(25)$$

$$M_C = \frac{\mp \; \Sigma \, m \, y' \; - \; H_C \, \Sigma \, y \, y'}{\Sigma \, y'} \dots\dots\dots\dots (18)$$

$$V_C = \frac{m_A - M_C - H_C \, y_1}{x_1} \dots\dots\dots\dots (26)$$

Temperature.—In this case,

$$M_L = M_C + H_C \, y + V_C \, x \dots\dots\dots\dots (27)$$

$$M_R = M_C + H_C \, y - V_C \, x \dots\dots\dots\dots (28)$$

Also, for the hinged end,

$$M_A = M_C + H_C \, y_1 + V_C \, x_1 = 0 \dots\dots\dots\dots (29)$$

As for earth and water pressure, $\Sigma \, M \, y'$ and $\Sigma \, M \, x' = 0$, so that:

$$M_C \, \Sigma \, y' + H_C \, \Sigma \, y \, y' = \mp \; \frac{I \, E \, a \, t \, L}{d \, s} \dots\dots\dots\dots (30)$$

$$M_C \, \Sigma \, x' + H_C \, \Sigma \, y \, x' - V_C \, \Sigma \, x^2 = 0 \dots\dots\dots\dots (31)$$

Solving Equations (28), (29), and (30), simultaneously,

$$H_C = \frac{\mp \; \dfrac{I \, E \, a \, t \, L}{d \, s \; \Sigma \, y'} \left(\dfrac{\Sigma \, x^2}{x_1} + \Sigma \, x' \right)}{\dfrac{\Sigma \, y \, y' \; \Sigma \, x'}{\Sigma \, y'} - \Sigma \, y \, x' + \dfrac{\Sigma \, x^2 \; \Sigma \, y \, y'}{x_1 \; \Sigma \, y'} - \dfrac{y_1 \; \Sigma \, x^2}{x_1}} \dots\dots (32)$$

$$M_C = \frac{\mp \; I \, E \, a \, t \, L - H_C \, \Sigma \, y \, y'}{\Sigma \, y'} \dots\dots\dots\dots (33)$$

$$V_C = - \frac{M_C + H_C \, y_1}{x_1} \dots\dots\dots\dots (34)$$

As previously, use the lower sign for temperature decrease, the upper one for increase.

Rib-Shortening.—As before, replace $E \, a \, t$ in Equations (32) and (33) with C, and solve as for temperature stress.

DOUBLE ARCH

There are six possible combinations of hinging and fixing the double arch (Fig. 8), as follows:
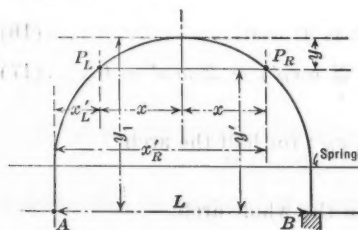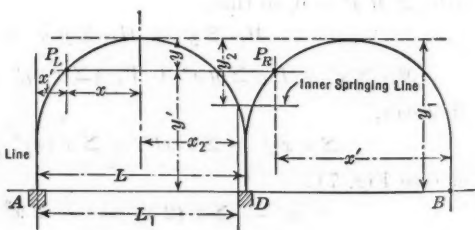
FIG. 7.   FIG. 8.

Symmetrical bearing:

       I.—All abutments fixed.

       II.—Outside abutments fixed, center abutment hinged.

       III.—All abutments hinged.

       IV.—Outside abutments hinged, center abutment fixed.

Unsymmetrical bearing:

V.—Left and center abutment fixed, right abutment hinged.

VI.—Left and center abutment hinged, right abutment fixed.

Earth and Water Pressure.—Now, in all combinations:

$$M \text{ (for the left arch)} = M_A + (M_D - M_A) \frac{x'}{L} + H_A y' + V_A x' \pm \mu_L. \quad (35)$$

$$M \text{ (for the right arch)} = M_B - (M_D + M_B) \frac{x'}{L} + H_B y' + V_B x' \pm \mu_R. \quad (36)$$

For the six combinations under unsymmetrical loading there will be some horizontal movement of the two inner springing lines (see Fig. 9). However, $\Sigma M_L y' \frac{d s}{E I}$ (horizontal movement of left arch) may be valued as equal and of opposite sign to $\Sigma M_R y' \frac{d s}{E I}$. Thus,

$$\Sigma M_L y' + \Sigma M_R y' = 0$$
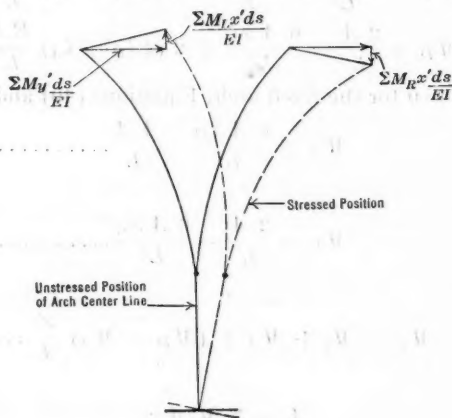
FIG. 9.

or,

$$(M_A + M_B) \left(\Sigma y' - \frac{\Sigma x' y'}{L_1} \right) + (H_A + H_B) \Sigma (y')^2$$
$$+ (V_A + V_B) \Sigma x' y' \pm \Sigma \mu_L y' \pm \Sigma \mu_R y' = 0 \dots \dots (37)$$

Also, from Fig. 9, the vertical displacement of one inner springing line relative to the other is an extremely small fraction of $\Sigma M_L y' \frac{d s}{E I}$ or $\Sigma M_R y' \frac{d s}{E I}$, or is practically zero, so that

$$\Sigma M_L x' = 0$$

or,

$$M_A \left(\Sigma x' - \frac{\Sigma (x')^2}{L_1} \right) + H_A \Sigma x' y' + V_A \Sigma (x')^2$$
$$+ M_D \frac{\Sigma (x')^2}{L_1} \pm \Sigma \mu_L x' = 0 \dots \dots \dots (38)$$

and,

$$\Sigma\, M_R\, x' = 0$$

or,

$$M_B \left(\Sigma\, x' - \frac{\Sigma\, (x')^2}{L_1} \right) + H_B\, \Sigma\, x'\, y' + V_B\, \Sigma\, (x')^2$$

$$- M_D\, \frac{\Sigma\, (x')^2}{L_1} \pm \Sigma\, \mu_R\, x' = 0 \dots\dots\dots\dots\dots (39)$$

For the double arch, there are seven forces to be determined, namely, M_A, H_A, V_A, M_D, M_B, V_B, and H_B. Since Equations (37), (38), and (39) hold for all six combinations of bearing, there need to be developed, in each case, four equations, which, when solved simultaneously with Equations (37), (38), and (39), will supply the values of the seven forces for the various combinations of bearing. These sets of four equations are developed as follows:

Combination I.—From Morley*:

$$M_A = \frac{6\, A\, x_G}{L^2} - \frac{4\, A}{L} - 2\, (i_D + 2\, i_A)\, \frac{E\, I}{L} \dots\dots\dots\dots (40)$$

$$M_D = \frac{2\, A}{L} - \frac{6\, A\, x_G}{L^2} + 2\, (2\, i_D + i_A)\, \frac{E\, I}{L} \dots\dots\dots\dots (41)$$

Since i_A and $i_D = 0$ for the fixed arch, Equations (40) and (41) reduce to:

$$M_A = \frac{6\, A\, x_G}{L^2} - \frac{4\, A}{L} \dots\dots\dots\dots\dots\dots (42)$$

and,

$$M_D = \frac{2\, A}{L} - \frac{6\, A\, x_G}{L^2} \dots\dots\dots\dots\dots\dots (43)$$

From Morley,†

$$M_L = M_1 + M_A + (M_D - M_A)\, \frac{x'}{L} \dots\dots\dots\dots (44)$$

Also,

$$A = \Sigma\, M_1\, d\, x$$

and,

$$A\, x_G = \Sigma\, M_1\, x'\, d\, x$$

Substituting these values of A and $A\, x_G$ in Equations (42) and (43); substituting the resulting values of M_A and M_D in Equation (44); and substituting for M_L its value as of Equation (35),

$$M_A = \frac{6}{L^2}\, (H_A\, \Sigma\, x'\, y'\, d\, x + V_A\, \Sigma\, (x')^2\, d\, x \pm \Sigma\, \mu_L\, x'\, d\, x)$$

$$- \frac{4}{L}\, (H_A\, \Sigma\, y'\, d\, x + V_A\, \Sigma\, x'\, d\, x \pm \Sigma\, \mu_L\, d\, x) \dots\dots\dots (45)$$

$$M_D = - \frac{6}{L^2}\, (H_A\, \Sigma\, x'\, y'\, d\, x + V_A\, \Sigma\, (x')^2\, d\, x \pm \Sigma\, \mu_L\, x'\, d\, x)$$

$$+ \frac{2}{L}\, (H_A\, \Sigma\, y'\, d\, x + V_A\, \Sigma\, x'\, d\, x \pm \Sigma\, \mu_L\, d\, x) \dots\dots\dots (46)$$

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* "Strength of Materials", p. 203.
† *Loc. cit.*, p. 201.

Also, for the right arch:

$$M_B = \frac{6}{L^2} (H_B \, \Sigma \, x' \, y' \, dx + V_B \, \Sigma \, (x')^2 \, dx \pm \Sigma \, \mu_R \, x' \, dx)$$

$$- \frac{4}{L} (H_B \, \Sigma \, y' \, dx + V_B \, \Sigma \, x' \, dx \pm \Sigma \, \mu_R \, dx) \dots\dots\dots (47)$$

$$M_D = \frac{6}{L^2} (H_B \, \Sigma \, x' \, y' \, dx + V_B \, \Sigma \, (x')^2 \, dx \pm \Sigma \, \mu_R \, x' \, dx)$$

$$- \frac{2}{L} (H_B \, \Sigma \, y' \, dx + V_B \, \Sigma \, x' \, dx \pm \Sigma \, \mu_R \, dx) \dots\dots\dots (48)$$

Equations (45), (46), (47), and (48), form the set of four equations required for Combination I.

Combination II.—Since $i_{tA} = 0$ and $M_D = 0$,

$$M_A = 6 \, A \, x_G - \frac{4 \, A}{L} - 2 \, i_D \, \frac{E \, I}{L} \dots\dots\dots\dots (49)$$

and,

$$M_D = \frac{2 \, A}{L} - \frac{6 \, A \, x_G}{L^2} + \frac{4 \, i_D \, E \, I}{L} = 0 \dots\dots\dots\dots (50)$$

from Equations (40) and (41).

Solving Equations (49) and (50) simultaneously,

$$M_A = \frac{3 \, A \, x_G}{L^2} - \frac{3 \, A}{L} \dots\dots\dots\dots\dots (51)$$

Substituting values of A and $A \, x_G$ as for Combination I, in Equation (51); substituting the resulting value of M_A in Equation (44); and substituting for M_L its value as of Equation (35):

$$M_A = \frac{3}{L^2} (H_A \, \Sigma \, x' \, y' \, dx + V_A \, \Sigma \, (x')^2 \, dx \pm \Sigma \, \mu_L \, x' \, dx)$$

$$- \frac{3}{L} (H_A \, \Sigma \, y' \, dx + V_A \, \Sigma \, x' \, dx \pm \Sigma \, \mu_L \, dx) \dots\dots\dots (52)$$

Similarly, for the right arch,

$$M_B = \frac{3}{L^2} (H_B \, \Sigma \, x' \, y' \, dx + V_B \, \Sigma \, (x')^2 \, dx \pm \Sigma \, \mu_R \, x' \, dx)$$

$$- \frac{3}{L} (H_B \, \Sigma \, y' \, dx + V_B \, \Sigma \, x' \, dx \pm \Sigma \, \mu_R \, dx) \dots\dots\dots (53)$$

Since the outside abutments are both fixed, $\Sigma \, M_L$ is equal and of opposite sign to $\Sigma \, M_R$, so that:

$$\Sigma \, M_L + \Sigma \, M_R = 0$$

or,

$$(M_A + M_B) \left(N - \frac{\Sigma \, x'}{L_1} \right) + (H_A + H_B) \, \Sigma \, y' + (V_A + V_B) \, \Sigma \, x'$$

$$\pm (\Sigma \, \mu_L + \Sigma \, \mu_R) = 0 \dots\dots\dots\dots\dots (54)$$

and,

$$M_D = 0 \dots\dots\dots\dots\dots\dots\dots\dots (55)$$

Equations (52), (53), (54), and (55), form the set of four equations required for Combination II.

Combination III.—Since $M_A = 0$ and $M_D = 0$ (for the hinged condition),

$i_D = \dfrac{3\,A\,x_G}{L\,E\,I}$ from Equations (40) and (41); but, $M_B = 0$ and $M_D = 0$, so that

again i_D (loading the right arch) $= -\dfrac{3\,A\,x_G}{L\,E\,I}$.

Since i_D for the left arch loading is equal and of opposite sign to i_D for the right arch loading,

$$A\,x_G \text{ (left arch)} = A\,x_G \text{ (right arch)}$$

or,

$$H_A\,\Sigma\,x'\,y'\,d\,x + V_A\,\Sigma\,(x')^2\,d\,x + \Sigma\,\mu_L\,x'\,d\,x = H_B\,\Sigma\,x'\,y'\,d\,x$$
$$+ V_A\,\Sigma\,(x')^2\,d\,x + \Sigma\,\mu_R\,x'\,d\,x \dots \dots \dots \dots \dots (56)$$

Also,

$$M_A = 0 \dots \dots \dots \dots \dots \dots \dots \dots (57)$$
$$M_B = 0 \dots \dots \dots \dots \dots \dots \dots \dots (58)$$

and

$$M_D = 0 \dots \dots \dots \dots \dots \dots \dots \dots (55)$$

Equations (55), (56), (57), and (58) form the set of four equations required for Combination III.

Combination IV.—Since, in this case, $M_A = 0$ and $i_B = 0$, $M_D = -\dfrac{3\,A\,x_G}{L^2}$

from Equations (40) and (41), or substituting as before,

$$M_D = -\frac{3}{L^2}\,(H_A\,\Sigma\,x'\,y'\,d\,x + V_A\,\Sigma\,(x')^2\,d\,x \pm \Sigma\,\mu_L\,x'\,d\,x)\dots(59)$$

Similarly, for the right arch,

$$M_D = \frac{3}{L^2}\,(H_B\,\Sigma\,x'\,y'\,d\,x + V_B\,\Sigma\,(x')^2 + \Sigma\,\mu_R\,x'\,d\,x)\dots\dots(60)$$

Also,

$$M_A = 0 \dots \dots \dots \dots \dots \dots \dots \dots (57)$$

and,

$$M_B = 0 \dots \dots \dots \dots \dots \dots \dots \dots (58)$$

Equations (57), (58), (59), and (60) form the set of four equations required for Combination IV.

Combination V.—As for Combination I, Equations (45) and (46) are available for the left arch; and as for Combination IV, Equations (58) and (60) may be used for the right arch. Equations (45), (46), (58), and (60) then form the set of four equations required for Combination V.

Combination VI.—As for Combination II, Equation (52) is useful for the left arch; and as for Combination III, i_D for the right arch $= -\dfrac{3\,A\,x_G}{L\,E\,I}$. From

Equation (50), i_D for the left arch $= \dfrac{3\,A\,x_G}{2\,L\,E\,I} - \dfrac{A}{2\,E\,I}$; and since i_D for the

left arch loading is equal and of opposite sign to i_D for the right arch loading:

$$\frac{3\,A\,x_G}{2\,L\,E\,I} - \frac{A}{2\,E\,I} \text{ (for left arch loading)} = \frac{3\,A\,x_G}{L\,E\,I} \text{ (for right arch loading)}$$

or,

$$\frac{3}{2\,L^2}\left(H_A\,\Sigma\,x'\,y'\,d\,x + V_A\,\Sigma\,(x')^2\,d\,x \pm \Sigma\,\mu_L\,x'\,d\,x\right)$$

$$-\frac{1}{2\,L}\left(H_A\,\Sigma\,y'\,d\,x + V_A\,\Sigma\,x'\,d\,x \pm \Sigma\,\mu_L\,d\,x\right)$$

$$=\frac{3}{L^2}\left(H_B\,\Sigma\,x'\,y'\,d\,x + V_B\,\Sigma\,(x')^2\,d\,x \pm \Sigma\,\mu_R\,x'\,d\,x\right)\ldots\ldots\ldots(61)$$

Also,

$$M_D = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(55)$$

and,

$$M_B = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(58)$$

Equations (52), (55), (58), and (61) form the set of four equations required for Combination VI.

In all equations for the double arch, Equations (45) to (61), use the lower sign for earth pressure; the upper one for water pressure.

Temperature.—Corresponding to the six combinations of bearing, for earth and water pressure, there are identical sets of formulas for temperature stress, with the following exceptions:

1.—Change the last two terms in Equation (37) from $\pm \Sigma \mu_L\,y' \pm \Sigma \mu_R y'$ to $\pm \dfrac{I\,E\,a\,t\,L}{d\,s}$.

2.—In all the other equations used for the solution of the seven forces $(M_A, H_A, V_A, M_D, M_B, H_B, V_B)$ omit all terms containing μ, such as, $\Sigma\mu$, $\Sigma\mu dx$, $\Sigma\mu x' dx$, etc.

Note that for temperature, the lower sign is to be used for temperature decrease and the upper one for increase.

Rib-Shortening.—As for the single arch, replace the term, $E\,a\,t$, with C and use the same formulas as for temperature stress in the double arch, for the solution of M_A, H_A, V_A, M_D, M_B, H_B, and V_B.

In Equations (45), (46), (47), etc., the summations nominally extend from an outside abutment to the abutment of the center wall. However, in all such cases the summation contains dx as a factor, which is zero for all center wall voussoirs. These summations, therefore, may be considered as actually extending from an outside abutment to the corresponding inner springing line.

DESIGN OF THE ARCH

General Conditions.—The stresses were computed by the method given by Metcalf and Eddy.* Working stresses, in pounds per square inch, were used as follows: Concrete, $f_c = 540$; steel, $f_s = 16\,000$; steel in compression, $f_s' = 8\,100$; shear $= 40$; bond $= 80$; punching shear on footing $= 150$; and $n = 15$. The thrust at the sides of the arch was considered equal to the full vertical reaction, as a measure of safety. The depth to the steel throughout the

* "American Sewerage Practice", Vol. 1, pp. 502–506.

arch was made 3 in. Section S-1 is considered fixed at the bottom of each footing; Section S-2 is considered fixed just below the springing lines. Sections S-3 and S-4 are considered hinged at the bottom of each footing. (Where larger values are obtained by the condition of fixing one end and hinging the other, these are used in preference.) Sections D-1 and D-2 are considered hinged at the bottom of each footing. (Where larger values are obtained by the condition of fixing one end and the center wall and hinging the other, these are used in preference.) Sections D-3 and D-4 are considered fixed just below the outside springing line.

Calculations for Typical Sections.—A single-barrel arch section, 1 ft. long, 18 in. thick at the crown, and 24 in. thick at the springing line and invert, was analyzed for the thirteen conditions (*A* to *M*) of earth loading and restraint (Table 1); for the maximum full content of water in the sewer, with the three conditions of restraint; and for a change in temperature of 50° Fahr. with the three conditions of restraint. Fills over the crown were taken as 2 ft. and 10 ft. Since, for a given arch, all linear quantities, such as *x* and *y*, etc., are constant, moments and thrusts will vary in a straight line with all positive fills over the crown. Thus, moments and thrusts for various fills were found by interpolation from the values for 2 ft. and 10 ft.

A double-barrel arch section of similar dimensions was analyzed for various conditions of restraint—for symmetrical earth loading of 0 and 10-ft. fills, four different water heads, and a temperature change of 50° Fahr.

In addition, the single and double arches encased in side-walls of sufficient height to insure complete fixation were analyzed with shortened arch rings, the rings in this case being considered as sprung from the top of the rock side-wall. The results of these computations are shown in Tables 2 to 5, inclusive.

Design of Spread Footing.—In the single-section sewer, as the fills became greater the soil became firmer, so that the bearing value of 2½ tons per sq. ft. could be increased and the projection of the spread footing past the arch ring became so small that bending could almost be neglected. Bending the vertical wall bars and extending them into the footing took care of any possible moment. The projection was made thick enough to be ample in resisting punching shear.

Extent of Various Sections.—From the profile, the 32-ft. single-barrel sewer was separated into four sections and the 29-ft. double-barrel sewer into three sections, as indicated in Table 6.

General Formulas for the Semi-Circular Sewer Arch

The semi-circular arch with side-walls of heights equal to one-third the radius is a standard section for St. Louis sewers. It was found feasible to derive formulas for thrust, shear, and moment for earth, water, and temperature pressure at critical points in the arch for various types of restrained ends.

Now, for two such arches, if their earth-fills and water heads are proportional, the bending moments at corresponding points throughout their rings will vary as the cubes of their radii. Similarly, the thrusts and shears will

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TABLE 2.—SINGLE ARCH. MOMENTS, IN THOUSANDS OF FOOT-POUNDS, DUE TO VARIOUS LOADINGS AND CONDITIONS OF RESTRAINT (SEE TABLE 1).

I. — MOMENTS DUE TO EARTH PRESSURE, FOR 2-FT. AND 10-FT. DEPTHS OF FILL OVER CROWN AND TO THIRTEEN CONDITIONS OF LOADING.

Point	Loading	Condition A.	Condition B.	Condition C.	Condition D.	Condition E.	Condition F.	Condition G.	Condition H.	Condition I.	Condition J.	Condition K.	Condition L.	Condition M.
Crown	2-ft. earth-fill	+0.2	+1.6	−1.4	+0.9	−1.7	+5.0	−6.8	+5.0	−15.2	−2.2	+0.2	−2.8	−0.1
	10-ft. earth-fill	+10.5	+11.7	−1.5	+11.2	+14.3	+23.8	−9.4	+23.8	+20.9	+18.5	+0.2	−13.4	−6.8
Near springing line	2-ft. earth-fill	−2.5	−5.8	+6.6	−5.3	+1.2	−18.4	+54.1	−18.2	+50.1	−1.0		−1.8	+1.0
	10-ft. earth-fill	−16.1	−20.8	+9.8	−19.5	−20.8	−39.2	+74.7	−40.0	+69.8	−22.7		−23.1	−9.2
End of arch	2-ft. earth-fill	−9.7	−16.6	−58.9	−19.7	0	0	0	+0.6	+80.5	+5.0	−71.8	+40.9	+4.5
	10-ft. earth-fill	+17.2	+53.0	−81.4	+57.6	0	0	0	+1.6	+111.3	+6.3	−91.8	+55.2	+25.4

II. — MOMENTS DUE TO VARIOUS HEADS OF WATER AT CROWN, AND TO TEMPERATURE AND RIB-SHORTENING.

Point	Loading	Fixed at Both Ends, A.				Fixed at Both Ends, B.				Hinged at Both Ends.				Hinged at One End, Fixed at Other End.			
		+4-ft.	0-ft.	−4-ft.	−8-ft.	+4-ft.	0-ft.	−4-ft.	−8-ft.	+4-ft.	0-ft.	−4-ft.	−8-ft.	+4-ft.	0-ft.	−4-ft.	−8-ft.
Crown	Water	+10.7	+10.2	+5.6	+2.3	+5.1	+5.0	+2.0	+0.5	+24.5	+22.6	+14.8	+7.9	+25.7	+23.0	+15.9	+8.4
	Temperature and rib-shortening	±2.9	±2.9	±2.9	±2.9	±6.2	±6.2	±6.2	±6.2	±1.2	±1.2	±1.2	±1.2	±1.9	±1.9	±1.9	±1.9
Near springing line	Water	−15.0	−14.3	−9.8	−4.4	+6.7	+6.7	−3.4	−0.7	−40.3	−37.1	−27.4	−15.4	−47.2	−43.6	−32.6	−18.2
	Temperature and rib-shortening																
End of arch	Water	+67.9	+61.3	+45.2	+27.2	+20.7	+19.9	+12.5	+5.0	0	0	0	0	±6.6	±6.6	±6.6	±6.6
	Temperature and rib-shortening	±7.6	±7.6	±7.6	±7.6	±15.4	±15.4	±15.4	±15.4	0	0	0	0	±6.6	±6.6	±6.6	±6.6

TABLE 3.—DOUBLE ARCH. MOMENTS, IN THOUSANDS OF INCH-POUNDS, DUE TO EARTH PRESSURE, WATER PRESSURE, TEMPERATURE, AND RIB-SHORTENING. VARIOUS CONDITIONS OF RESTRAINT, VARIOUS DEPTHS OF FILL, AND VARIOUS HEADS OF WATER AT CROWN.

Point	Loading	ALL ABUTMENTS FIXED AT THEIR BASES			ALL ABUTMENTS FIXED AT THE ARCH SPRINGING LINE				LEFT AND RIGHT ABUTMENTS HINGED AT THEIR BASES; CENTER WALL FIXED AT BASE				LEFT ABUTMENT FIXED AT BASE; RIGHT ABUTMENT HINGED AT BASE; CENTER WALL FIXED AT BASE			
		+5 ft. head of water, 0 ft. earth-fill	−5 ft. head of water, 10-ft. earth-fill	−10 ft. head of water, 10-ft. earth-fill	+5 ft. head of water, 0 earth-fill	0 ft. head of water, 0 earth-fill	−5 ft. head of water, 10-ft. earth-fill	−10 ft. head of water, 10 ft. earth-fill	+5 ft. head of water, 0 earth-fill	0 ft. head of water, 0 earth-fill	−5 ft. head of water, 10-ft. earth-fill	−10 ft. head of water, 10-ft. earth-fill	+5 ft. head of water, 0 earth-fill	0 ft. head of water, 0 earth-fill	−5 ft. head of water, 10-ft. earth-fill	−10 ft. head of water, 10-ft. earth-fill
Crown { Earth		− 0.9	+ 8.2	+ 8.2	− 0.3	− 0.3	+ 7.1	+ 7.1	− 2.6	− 2.6	+11.9	+11.9	− 0.7	− 0.7	+10.1	+10.1
Water		+ 5.6	+ 1.9	+ 0.4	+ 2.7	+ 2.8	+ 0.7	0	+14.1	+12.9	+ 4.3	+ 1.5	+10.9	+10.1	+ 4.8	+ 1.5
Temperature and rib-shortening		± 5.0	± 5.0	± 5.0	± 6.5	± 6.5	± 6.5	± 6.5	± 1.8	± 1.8	± 1.8	± 1.8	± 3.6	± 3.6	± 3.6	± 3.6
Near outside springing line { Earth		− 0.1	−13.3	−13.3	+ 2.1	+ 2.1	− 5.1	− 5.1	+ 2.9	+ 2.9	−28.8	−28.8	+ 5.6	+ 5.6	− 2.0	−28.0
Water		− 8.5	− 5.1	− 1.1	+ 7.2	+ 5.5	− 1.6	− 0.2	−29.7	−27.7	−16.7	− 6.2	−44.1	−35.2	−20.9	− 8.6
Temperature and rib-shortening		± 1.8	± 1.8	± 1.8	± 2.4	± 2.4	± 2.4	± 2.4	± 2.8	± 2.8	± 2.8	± 2.8	± 5.6	± 5.6	± 5.6	± 5.6
End of arch { Earth		−10.5	+25.8	+25.8	− 0.9	− 0.9	+25.9	+25.9	0	0	0	0	......	......	......	......
Water		+48.0	+28.6	+15.2	+19.2	+19.3	+ 9.6	+ 1.9	0	0	0	0	......	......	......	......
Temperature and rib-shortening		±20.3	±20.3	±20.3	±21.5	±21.5	±21.5	±21.5	0	0	0	0	......	......	......	......
Inside springing line { Earth		+ 1.0	−12.9	−12.9	− 6.2	− 6.2	−22.5	−22.5	+ 2.9	+ 2.9	−21.4	−21.4	+ 1.8	+ 1.8	−19.3	−19.3
Water		− 9.5	− 4.7	− 1.0	+ 4.4	− 0.1	− 1.7	− 0.2	−27.0	−21.1	−12.3	− 3.0	−23.5	−23.6	−16.2	− 4.9
Temperature and rib-shortening		±14.4	±14.4	±14.4	±17.1	±17.1	±17.1	±17.1	±11.0	±11.0	±11.0	±11.0	±10.0	±10.0	±10.0	±10.0

TABLE 4.—Single Arch, Moments, Thrusts, Shears, and Concrete Design.

Section.*	Earth-fill, in feet.	Water head, in feet.	Critical sections.	Moment, in thousands of foot-pounds.	Thrust, in thousands of pounds.	Shear, in thousands of pounds.	Full depth, in inches.	$\frac{Q+}{t}$	p.	k.	C_A.	f_c, in pounds.	f_s, in pounds.	f'_s, in pounds.	Shear, in pounds.	Bond, in pounds.	A_s, on each side.	Reinforcing steel on each side.
S-1	15	0	Crown	+48	18		27	1.64	0.0082	0.305	0.126	525	15 700	O.K.			1.83	1 in. square 9 in., c. to c.
			Sides	+76	45	12½	36	0.56	0.0061	0.435	0.111	530	9 000	O.K.	36	41	1.83	1 in. square 9 in., c. to c.
S-2	20	0	Crown	+27	18		29	0.86	0.0061	0.345	0.113	510	13 800	O.K.			0.84	⅞ in. square 11 in., c. to c.
			Sides	+76	55	15	33	0.42	0.0086	0.435	0.096	530	8 900	O.K.	40	66	0.84	⅞ in. square 11 in., c. to c.
S-3	10	4	Crown	+46	8		24	2.87	0.0117	0.315	0.149	535	14 500	O.K.			1.69	1¼ in. square 9 in., c. to c.
			Sides	+73	35	10	32	0.78	0.0088	0.385	0.127	560	10 800	O.K.	33	27	1.69	1¼ in. square 9 in., c. to c.
S-4	5	2	Crown	—32	5		21	3.65	0.0106	0.290	0.146	490	14 700	O.K.			1.83	1 in. square 9 in., c. to c.
			Sides	—56	25	8	28	0.96	0.0100	0.380	0.134	535	12 500	O.K.	30	25	1.69	1¼ in. square 9 in., c. to c.

* See Table 6.   † Q = the moment, in inch-pounds, divided by the product of the thrust, in pounds, and the full depth of beam, in inches.

TABLE 5.—Double Arch, Moments, Thrusts, Shears, and Concrete Design.

Section.*	Earth-fill, in feet.	Water head, in feet.	Critical sections.	Moment, in thousands of foot-pounds.	Thrust, in thousands of pounds.	Shear, in thousands of pounds.	Full depth, in inches.	$\frac{Q+}{t}$	p.	k.	C_A.	f_c, in pounds.	f_s, in pounds.	f'_s, in pounds.	Shear, in pounds.	Bond, in pounds.	A_s, on each side.	Reinforcing steel on each side.
D-1	5	5	Crown	+22	—2		18	7.35	0.0095	0.270	0.188	495	16 200	O.K.			1.02	⅞ in. square 9 in., c. to c.
			Sides	+58	16	5	30	1.45	0.0074	0.305	0.119	540	16 200	O.K.	16	20	1.33	1 in. square 9 in., c. to c.
			Inside	+25	13	6	27	1.64	0.0082	0.305	0.126	525	15 700	O.K.	21	27	1.33	1 in. square 9 in., c. to c.
D-2	10	0	Crown	—67	—3		18	5.50	0.0110	0.290	0.150	515	15 300	O.K.			1.19	⅞ in. square 8 in., c. to c.
			Sides	—52	28	5	30	0.96	0.0108	0.385	0.139	535	9 400	O.K.	16	16	1.95	1¼ in. square 8 in., c. to c.
			Inside	+21	24	6	27	0.98	0.0120	0.400	0.145	495	9 000	O.K.	21	21	1.95	1¼ in. square 8 in., c. to c.
D-3	15	0	Crown	+75	15		18	0.69	0.0080	0.360	0.122	530	10 600	O.K.			1.40	1⅛ in. square 11 in., c. to c.
			Sides	+46	36	10	36	0.55	0.0065	0.385	0.112	520	11 000	O.K.	27	36	1.87	1⅛ in. square 11 in., c. to c.
			Inside	+24	35	7	30	0.80	0.0048	0.415	0.108	520	8 700	O.K.	23	39	0.87	⅞ in. square 11 in., c. to c.
D-4	20	0	Crown	—88	20		18	0.65	0.0110	0.415	0.139	535	8 000	O.K.			1.19	⅞ in. square 8 in., c. to c.
			Sides	—55	45	12	36		0.0090	0.435	0.127	535	9 000	O.K.	32	31	1.95	1¼ in. square 8 in., c. to c.
			Inside		44	8	30	0.50	0.0066	0.465	0.114	535	8 000	O.K.	25	32	1.19	⅞ in. square 8 in., c. to c.

* See Table 6.   † Q = the moment, in inch-pounds, divided by the product of the thrust, in pounds, and the full depth of beam, in inches.

vary as the squares of their radii; and, since for any given arch, a simple formula can be found for these quantities with varying fills and water heads, a formula for combined variation of fill, water head, and radius may also be derived. As an example, the expression for the earth-pressure moment at the crown for a fixed single arch will be derived.

TABLE 6.—RIVER DES PERES SEWER.   SECTIONS FOR DESIGN.

	Section.	Earth-fill, in feet.	Water head, in feet.	Location.	Approximate length, in feet.
Single-barrel sewer ...	S-1	15	0	City limits to Delmar Avenue.	3 200
	S-2	20	0	Delmar to Rock Island Railroad.	2 000
	S-3	10	4	Rock Island Railroad to Government Drive.	4 000
	S-4	5	0	Government Drive to Lafayette Bridge.	2 100
Double-barrel sewer..	D-1	5	5	Lafayette Bridge to Clayton Road.	4 700
	D-2	10	0	Clayton Road to Oakland Avenue.	1 200
	D-3	15 to 20	0	Oakland Avenue to "Frisco" Railroad.	3 200

With a radius of 16 ft., the following moments would exist at the crown:

$$M \text{ for a 10-ft. fill} = 11\,000 \text{ ft-lb.}$$
$$M \text{ " " 5- " " } = 4\,400 \text{ " "}$$
$$M \text{ " " 0- " " } = -2\,200 \text{ " "}$$

That is, the moment, M, increases 6 600 ft-lb. for every 5-ft. increase in fill. Then, with a radius, R, of 32 ft., M will increase $6\,600 \times \left(\dfrac{32}{16}\right)^3$ ft-lb. for every 10-ft. increase in fill, F, or $6\,600 \times \left(\dfrac{32}{16}\right)^2$ for every 5-ft. increase in fill.*

Therefore, M, for any radius equals:

$$-2\,200 \left(\frac{R}{16}\right)^3 + 6\,600 \left(\frac{R}{16}\right)^2 \times \frac{F}{5}$$

which is approximately equal to $5 R^2 \left(F - \dfrac{R}{10} \right)$

For the case of water levels below the crown intrados, the variation in moment for a given radius is not uniform, but parabolic in shape, owing to the loss of loads along the curve of the intrados. Thus, the crown moment for single fixed arch (water pressure) $= 2\dfrac{1}{2} R^2 \left(R + \dfrac{H}{4} \right)$ above the crown intrados, and $2\dfrac{1}{2} R (R - K)^2$ below the crown intrados,† in which, H equals additional

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* The error introduced by assuming that the thickness of the arch ring is also proportional to the radius, is negligible.

† In all cases it was found safe to consider all water stresses equal to zero, when the water surface was no higher than the springing line of the arch.

TABLE 7.—Single Arch with Both Abutments Fixed.   Expressions for Moments, Thrusts, and Shears.
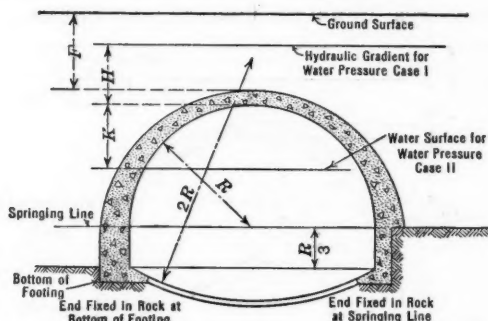

Fig. 10.

(Use Zero Values of Moment, Thrust, and Shear for Water Pressure, when Water Surface is below Springing Line ($K > R$); F, H, K, and R to be Calculated in Feet; t = Temperature Change.)

Point.	Loading.	Moment, in foot-pounds.	Thrust, in pounds.	Shear, in pounds.
		ARCH WITH ENDS FIXED AT BOTTOM OF FOOTING.		
Crown...	Earth.........	$+5 R^2 (F - 0.10 R)$	$+21 R (R + 3 F)$	0
	Water—I*.....	$+2.5 R^2 (R + 0.25 H)$	$-17 R (R + 3.80 H)$	0
	Water—II†...	$+2.5 R (R - K)^2$	$-17 (R - 1.25 K)^2$	0
	Temperature and rib-shortening..	$+0.23 t R^2$	$\pm 0.53 t R$	0
Bottom of footing....	Earth.........	$+13 R^2 (F - 0.31 R)$	$+66 R (R + 1.70 F)$	$+16.5 R (1.25 R - F)$
	Water—I*.....	$+15 R^2 (R + 0.33 H)$	$-13.5 R (R + 4.7 H)$	$-44 R (R + 0.50 H)$
	Water—II†...	$+15 R^2 (R - K)$	$-13.5 (R - 1.25 K)^2$	$-44 R (R - K)$
	Temperature and rib-shortening..	$+0.60 t R^2$	0	$\pm 0.53 t R$
		ARCH WITH ENDS FIXED AT SPRINGING LINE.		
Crown ..	Earth...... ..	$+3 R^2 (F - 0.14 R)$	$+20 R (R + 3.50 F)$	0
	Water—I*.....	$+1.25 R^3$	$-11.25 R (R + 5.60 H)$	0
	Water—II†...	$+1.25 R (R - K)^2$	$-11.25 (R - 1.50 K)^2$	0
	Temperature and rib-shortening..	$+0.48 t R^2$	$\pm 1.54 t R$	0
Springing line	Earth.........	$+10.25 R^2 (F - 0.02 R)$	$+66 R (R + 1.70 F)$	$-40 R (F - 0.03 R)$
	Water—I*.....	$+4.87 R^2 (R + 0.17 H)$	$-13.5 R (R + 4.7 H)$	$-24 R (R + 0.15 H)$
	Water—II†...	$+4.87 R^2 (R - 1.33 K)$	$-13.5 (R - 1.25 K)^2$	$-24 R (R - 1.13 K)$
	Temperature and rib-shortening..	$+1.20 t R^2$	0	$\pm 1.54 t R$

* Water—I means Water Pressure Case I (see Fig. 10).
† Water—II means Water Pressure Case II (see Fig. 10).

TABLE 8.—Single Arch with One or More Abutments Hinged. Expressions for Moments, Thrusts, and Shears.
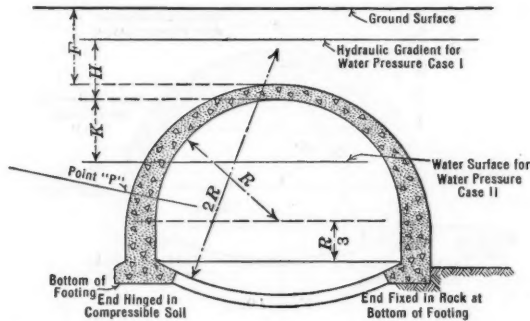

Fig. 11.

(Use Zero Values of Moment, Thrust, and Shear for Water Pressure when Water Surface is below Springing Line $(K > R)$; F, H, K, and R to be Calculated in Feet; $t =$ Temperature Change.)

Point.	Loading.	Moment, in foot-pounds.	Thrust, in pounds.	Shear, in pounds.
		ARCH HINGED AT BOTH ENDS.		
Crown...	Earth..............	$+8\ R^2\ (F - 0.17\ R)$	$+25\ R\ (R + 2\ F)$	0
	Water—I*...........	$+5.5\ R^2\ (R + 0.33\ H)$	$-28\ R\ (R + 2.5\ H)$	0
	Water—II†..........	$+5.5\ R\ (R - 0.75\ K)^2$	$-28\ (R - 0.90\ K)^2$	0
	Temperature and rib-shortening	$+0.09\ t\ R^2$	$\pm 0.06\ t\ R$	0
Point P...	Earth..............	$-11\ R^2\ (F - 0.17\ R)$	$+66\ R\ (R + 1.70\ F)$	$+17.5\ R\ (R - 0.36\ F)$
	Water—I*...........	$-9.1\ R^2\ (R + 0.33\ H)$	$-13.5\ R\ (R + 4.7\ H)$	$-32\ R\ (R + 0.50\ H)$
	Water—II†..........	$-9.1\ R^2\ (R - 1.13\ K)$	$-13.5\ (R - 1.25\ K)^2$	$-32\ R\ (R - 0.90\ K)$
	Temperature and rib-shortening	$-0.05\ t\ R^2$	0	$+0.06\ t\ R$
		ARCH HINGED AT ONE END, FIXED AT OTHER.		
Crown...	Earth..............	$+8\ R^2\ (F - 0.17\ R)$	$+25\ R\ (R + 2\ F)$	0
	Water—I*...........	$+5.75\ R^2\ (R + 0.33\ H)$	$-29\ R\ (R + 2.50\ H)$	0
	Water—II†..........	$+5.75\ R\ (R - 0.75\ K)^2$	$-29\ (R - 0.90\ K)^2$	0
	Temperature and rib-shortening..........	$+0.15\ t\ R^2$	$\pm 0.26\ t\ R$	0
Point P...	Earth..............	$-10.5\ R^2\ (F - 0.25\ R)$	$+66\ R\ (R + 1.70\ F)$	$+17\ R\ (R - 0.39\ F)$
	Water—I*...........	$-10.6\ R^2\ (R + 0.40\ H)$	$-13.5\ R\ (R + 4.7\ H)$	$-32\ R\ (R + 0.50\ H)$
	Water—II†ठ	$-10.6\ R^2\ (R - K)$	$-13.5\ (R - 1.25\ K)^2$	$-32\ R\ (R - 0.90\ K)$
	Temperature and rib-shortening.	$+0.30\ t\ R^2$	0	$+0.26\ t\ R$

* Water—I means Water Pressure Case I (see Fig. 11).
† Water—II means Water Pressure Case II (see Fig. 11).

TABLE 9.—Double Arch with One or More Abutments Hinged. Expressions for Moments, Thrusts, and Shears.
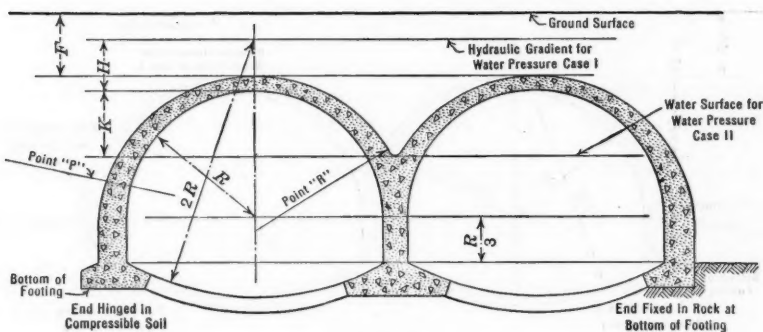

FIG. 12.

(F, H, K, and R to be Calculated in Feet; t = Temperature Change.)

Point.	Loading.	Moment, in foot-pounds.	Thrust, in pounds.	Shear, in pounds.
	Arch with All Abutments Hinged.			
Crown...	Earth.....................	$+7 R^2 (F - 0.12 R)$	$+25 R (R + 2 F)$	0
	Water—I*..............	$+4.2 R^2 (R + 0.33 H)$	$-27 R (R + 2.50 H)$	0
	Water—II†.............	$+4.2 R (R - K)^2$	$-27 (R - K)^2$	0
	Temperature and rib-shortening	$+0.17 t R^2$	$+0.37 t R$	0
Point P..	Earth.....................	$-11.5 R^2 (F - 0.08 R)$	$+66 R (R + 1.70 F)$	$+15.5 R (R - 0.50 F)$
	Water—I*..............	$-9.1 R^2 (R + 0.20 H)$	$-13.5 R (R + 4.7 H)$	$-31 R (R + 0.50 H)$
	Water—II†.............	$-9.1 R^2 (R - 1.13 K)$	$-13.5 (R - 1.25 K)^2$	$-31 R (R - K)$
	Temperature and rib-shortening	$-0.27 t R^2$	0	$\pm 0.37 t R$
Point R..	Earth.....................	$-11.5 R^2 (F - 0.08 R)$	$+50 R (R + 2.20 F)$	0
	Water—I*..............	$-6.9 R^2 (R + 0.88 H)$	$-10 R (R + 6.40 H)$	0
	Water—II†.............	$-6.9 R^2 (R - 1.17 K)$	$-10 (R - 1.50 K)^2$	0
	Temperature and rib-shortening	$-1.05 t R^2$	$\pm 0.19 t R$	$\pm 0.32 t R$
	Arch with One End Hinged, Other End Fixed, Center Wall Fixed.			
Crown...	Earth.....................	$+5 R^2 (F - 0.04 R)$	$+24 R (R + 2.30 F)$	0
	Water—I*..............	$+3.33 R^2 (R + 0.20 H)$	$-26 R (R + 2.50 H)$	0
	Water—II†.............	$+3.33 R (R - K)^2$	$-26 (R - K)^2$	0
	Temperature and rib-shortening	$+0.34 t R^2$	$+0.69 t R$	0
Point P..	Earth.....................	$-16 R^2 (F - 0.12 R)$	$+86 R (R + 1.70 F)$	$+16.5 R (R - 0.65 F)$
	Water—I*..............	$-11.5 R^2 (R + 0.75 H)$	$-13.5 R (R + 4.7 H)$	$-32 R (R + 0.60 H)$
	Water—II†.............	$-11.5 R^2 (R - 1.13 K)$	$-13.5 (R - 1.25 K)^2$	$-32 R (R - K)$
	Temperature and rib-shortening	$-0.54 t R^2$	0	$\pm 0.69 t R$
Point R..	Earth.....................	$-10 R^2 (F - 0.06 R)$	$+50 R (R + 2.20 F)$	0
	Water—I*..............	$-7.75 R^3$	$-10 R (R + 6.40 H)$	0
	Water—II†.....	$-7.75 R^2 (R - 1.06 K)$	$-10 (R - 1.50 K)^2$	0
	Temperature and rib-shortening	$-1.00 t R^2$	$\pm 0.35 t R$	$\pm 0.59 t R$

* Water—I means Water Pressure Case I (see Fig. 12).
† Water—II means Water Pressure Case II (see Fig. 12).

TABLE 10.—Double Arch with All Abutments Fixed. Expressions for Moments, Thrusts, and Shears.
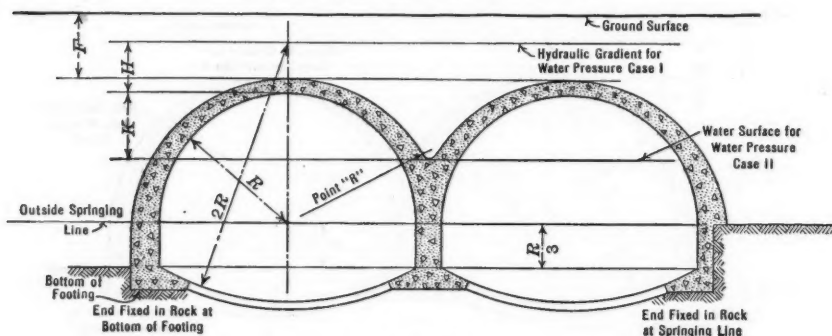

Fig. 13.

(F, H, K, and R to be Calculated in Feet; t = Temperature Change.)

Point.	Loading.	Moment, in foot-pounds.	Thrust, in pounds.	Shear, in pounds.
Arch with Both Ends Fixed at Bottom Footing; Center Wall Also.				
Crown.....	Earth..............	$+ 4.33 R^2 (F - 0.07) R$	$+21 R (R + 3 F)$	0
	Water—I*..........	$+ 1.75 R^2 (R + 0.17 H)$	$-14.5 R (R + 4.4 H)$	0
	Water—II†........	$+ 1.75 R (R - K)^2$	$-14.5 (R - 1.40 K)^2$	0
	Temperature and rib-shortening....	$+ 0.48 t R^2$	$+ 2.04 t R$	0
Bottom of footing..	Earth..............	$-17 R^2 (F - 0.20 R)$	$+66 R (R - 1.70 F)$	$+22 R (R - 1.20 F)$
	Water—I*..........	$+15.75 R^2 (R + 0.33 H)$	$-13.5 R (R + 4.7 H)$	$-43 R (R + 0.50 H)$
	Water—II†........	$+15.75 R^2 (R - K)$	$-13.5 (R - 1.25 K)^2$	$-43 R (R - K)$
	Temperature and rib-shortening....	$+ 1.93 t R^2$	0	$\pm 2.04 t R$
Point R....	Earth..............	$- 6.75 R^2 (F - 0.07 R)$	$+50 R (R + 2.20 F)$	0
	Water—I*..........	$- 2.75 R^2 (R + 0.40 H)$	$-10 R (R + 6.40 H)$	0
	Water—II†........	$- 2.75 R^2 (R - 1.30 K)$	$-10 (R - 1.50 K)^2$	0
	Temperature and rib-shortening....	$- 1.37 t R^2$	$\pm 1.02 t R$	$\pm 1.77 t R$
Arch with Both Ends Fixed at Springing Line; Center Wall at Bottom Footing.				
Crown.....	Earth..............	$+ 3.5 R^2 (F - 0.03 R)$	$+20 R (R + 3.50 F)$	0
	Water—I*..........	$+ 0.92 R^3$	$-10 R (R + 6 H)$	0
	Water—II†........	$+ 0.92 R (R - K)^2$	$-10 (R - 1.50 K)^2$	0
	Temperature and rib-shortening....	$+ 0.62 t R^2$	$+ 2.86 t R$	0
Outer springing line.......	Earth..............	$+12.75 R^2 (F - 0.02 R)$	$+66 R (R + 1.70 F)$	$-40 R (F - 0.03 R)$
	Water—I*..........	$+ 6.33 R^2$	$-13.5 R (R + 4.70 H)$	$-27 R (R + 0.25 H)$
	Water—II†........	$+ 6.33 R^2 (R - 1.83 K)$	$-13.5 (R - 1.25 K)^2$	$-27 R (R - 1.20 K)$
	Temperature and rib-shortening....	$+ 2.04 t R^2$	0	$+ 2.86 t R$
Point R....	Earth..............	$- 8 R^2 (F + 0.25 R)$	$+50 R (R + 2.20 F)$	0
	Water—I*..........	$+ 4.2 R^2 H$	$-10 R (R + 6.40 H)$	0
	Water—II†........	0	$-10 (R - 1.50 K)^2$	0
	Temperature and rib-shortening....	$\pm 1.63 t R^2$	$\pm 1.43 t R$	$\pm 2.48 t R$

* Water—I means Water Pressure Case I (see Fig. 13).

† Water—II means Water Pressure Case II (see Fig. 13).

height of gradient above the crown intrados, and K equals distance from the water surface to the crown intrados, when the sewer is only partly filled.

In the case of temperature stresses, for the fixed single arch:

$$M_C = \frac{I\,E\,a\,t\,L \times \Sigma\,y}{d\,s\left(\Sigma\,y^2 - \frac{(\Sigma y)^2}{N}\right)}$$

Now, I will vary approximately as the cube of the thickness of the arch ring, and this thickness will vary approximately as the square root of the moment. Therefore, I varies as the $\frac{3}{2}$ power of the moment. The quantities, $\Sigma\,y$ and $d\,s$, will vary as the span or radius, and the quantity, $\Sigma\,y^2 - \frac{(\Sigma y)^2}{N}$, as the square of the span or radius. Substituting for these quantities their values in terms of moment and span, there is obtained:

$$M_C = \frac{M^{\frac{3}{2}}\,E\,a\,t\,L\,k_1\,L}{k_2\,L\,.\,k_3\,L^2}, \text{ or } M_C = \frac{M_C^{\frac{3}{2}} \times k}{L}$$

in which, E, a, t, I, k_1, k_2, k_3, and k are constants. That is, M_C varies as the square of L or R. This relation holds equally for moments throughout the arch. By similar reasoning, the thrusts and shears will vary directly as the radius.

From these relations, Tables 7 to 10 for thrust, shear, and moment were compiled, by the use of which single and double arches of the standard interior section can be quickly designed after the radius, earth-fill, water head, temperature change, and character of abutment have been determined.

CONCLUSION

Although this paper attempts to present the hinged sewer arch as a definite and economical engineering structure, the fixation of one or more abutments is quite possible. Fixation of one end might occur when the rock plane has a considerable slope across the sewer line, or when the rock plane along one abutment line is above the abutment level, while the plane along the other line is several feet below. Fixation of both ends might occur with a more or less level rock plane above the abutment level. In all cases of fixation, however, the arch ring should be considered as shortened by the amount of side-wall required, in the judgment of the designer, to develop the fixing moment. For all loading (except a heavy horizontal earth pressure near the abutment in question) the tendency of the abutment is to sink on the extrados side, and rise on the intrados side. The fixing moment that resists this tendency needs to develop compression along the arch extrados and the jagged rock side-wall.

It is quite probable that in an actual sewer, neither complete fixation nor complete hinging of the abutments (as represented by the respective mathematical equations, $\Sigma\,M = 0$ and $M_A = 0$) are realized. However maximum moments for a partial fixation lie between and never outside those calculated for the completely fixed and those for the completely hinged arch. This fact strengthens the importance of considering the hinged arch, since it is only at

FIG. 14.

FIG. 15.

FIG. 16.

FIG. 17.
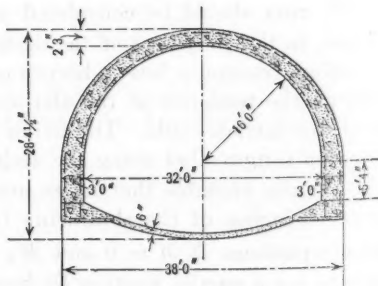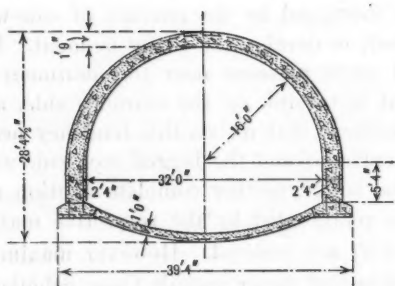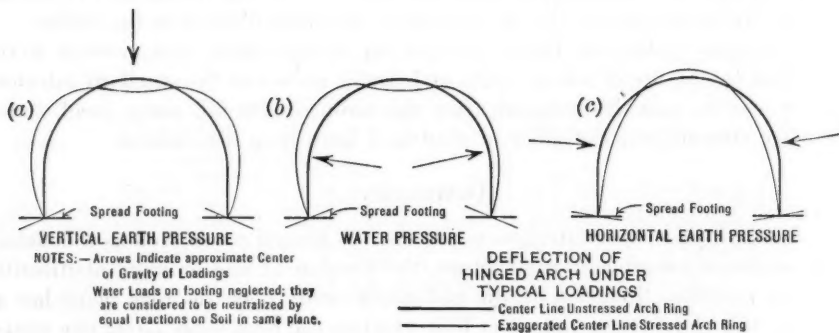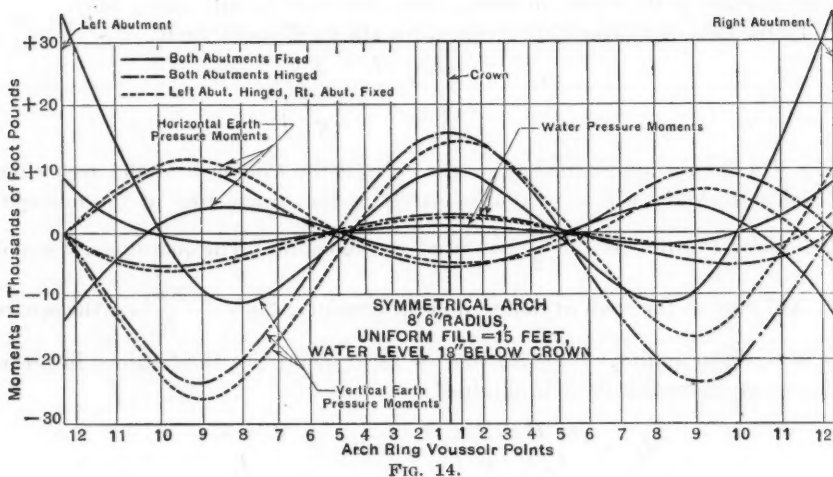
the fixed abutment that the bending moment for the fixed arch exceeds that for the hinged arch. Partial fixation, in all probability, reduces this excessive fixing moment to an amount equal to or less than the maximum sustained by the hinged arch.
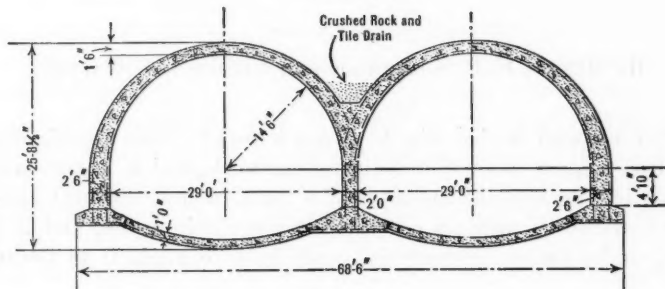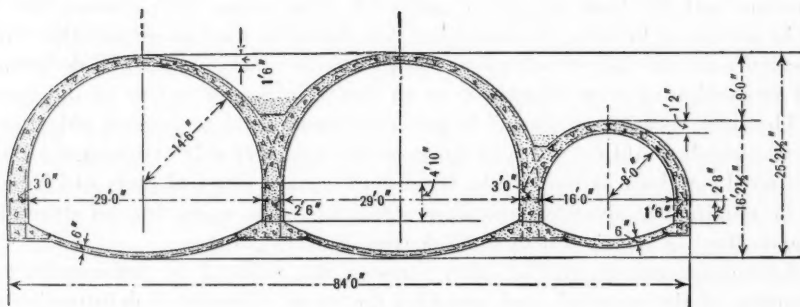

FIG. 18.

FIG. 19.

A typical bending moment variation throughout a single arch ring for the three conditions of arch support and the three main types of loading, is shown in Fig. 14.

In Fig. 15 is shown the deflection of a typical hinged arch under the three types of loading. It illustrates the co-ordinate bending due to vertical earth and water pressure, and the opposing effect of horizontal earth pressure.

Some of the sections to be used on the River Des Peres Drainage Works are given in Figs. 16, 17, 18, and 19. Figs. 16 and 17 represent the single arch which extends from the western city limits to the extension of Union Avenue in Forest Park. Fig. 18 represents the main double section through Forest Park, and Fig. 19 a special combination of the double arch with the Tower Grove Branch, which carries a short distance from Manchester Avenue to the upper end of the open-channel section.

SECONDARY STRESSES IN BRIDGES

Discussion*

By Messrs. F. P. Shearwood and Laurence J. Waller

F. P. Shearwood,† M. Am. Soc. C. E. (by letter).‡—This investigation must have involved a great amount of painstaking work, and it is very useful for correctly indicating how the members of a frame will be strained under load. The labor involved in working out these stresses is immense, and it is quite impossible, under present commercial conditions, to apply it to the ordinary structure.

The real aim of designing is to produce the safest, most rigid, and lasting structure with the least amount of material. Theoretical fiber stresses should not be permitted to befog the real issue, but should be used in conjunction with experience on existing structures to temper engineering judgment in designing and proportioning new structures or in determining the safety of old ones.

The term "secondary stress" is generally understood to include only those stresses which would not occur if the structure had perfectly articulated joints. This ideal condition is impossible, because of certain physical facts which cannot be eliminated; actually, secondary stresses exist in every framed structure because there is restriction to some degree in every joint.

Secondary stresses, unlike other stresses, are relieved by deformation or distortion of the material, and, provided the range of repeated deformation is not sufficient to cause fatigue, ordinary secondary stresses will not reduce the safety of the structure.

It was with great interest that the writer read Mr. Godfrey's discussion,§ with whom he agrees most thoroughly. Mr. Godfrey truly states that no steel structure has failed through the neglect of providing for secondary stresses. The failure of the counterweight truss members of the bascule bridges referred to by Mr. Godfrey‖ was attributed by some to secondary stresses; but no conclusive proof of this was given. A careful study of these designs would point to an equal probability of the failures being due to the tearing effect of the embedded trusses on the main members, because the combination of concrete and embedded steel had ceased to function. The breaking of the bond of the concrete was due to the frequent reversal of the stress.

Any engineer familiar with the fabrication and erection of steelwork must realize that even in the hands of the best bridge companies, the steel is subjected to greater strains than it can ever receive under its normal loading.

It seems peculiar that, although steel is the most ductile of all materials, the design of steel structures is practically the only one in which it is customary to neglect this quality deliberately. Indeterminateness is avoided and any form of continuity is unfavorably considered, because the supports might move and cause strains beyond the elastic limit, but, in practice, many examples of steel structures are found which are safely performing their duty in spite of some injury to their members that has distorted them far .beyond the ultimate strength of the material, if calculated according to the elastic theory. On the other hand, material such as concrete is used for all kinds of construction where continuity and other forms of indeterminateness exist in spite of the fact that these materials lack ductility.

To determine the effect of secondary stress from rigid joints on the safety of structures, it is necessary to know by experiment or practical experience whether or not restricted straining (secondary stress) really weakens members stressed in direct tension or compression in proportion to the increase of the theoretical fiber stress. Experience seems to indicate that it does not, in fact, there is much to show that secondary stress is more often a source of increased strength than of weakness.

LAURENCE J. WALLER,* Assoc. M. AM. Soc. C. E. (by letter).†—The author's work in compiling and comparing the various types of secondary stress analysis has placed before the profession an invaluable compendium of scientific information.

It has seemed to the writer that, in the case of the average engineer, the selection of a method of secondary stress calculation has been almost entirely fortuitous. The sources of information are not extensive; the subject itself is not in common usage, and is relatively difficult. Faced with a choice between a number of highly technical systems and having no basis of comparison other than actual trial, engineers have largely used the system nearest at hand, with the varying degrees of labor required and accuracy attained. The author's critique makes it possible for the engineer promptly to discard complicated or inherently faulty systems in favor of the simpler and more accurate types of analysis.

The writer considers Mohr's semi-graphical method to be the most practical thus far devised for the usual type of bridge truss. The precise details of this method, however, are subject to considerable variation. Personally, the writer uses an arrangement of Mohr's method so far not touched on in this discussion, namely, that of Mr. G. A. Maney.‡ The nomenclature and the arrangement are lucid and concise, and possess to an eminent degree the property of being readily visualized. Contrary to the author's preference, the solution by successive substitutions will probably commend itself to the profession rather than elimination by Gauss' normal equations, particularly in the case of long span bridges. Instead of the usual substitutions, a more rapid convergence of values is effected by using in a given equation the last

* Civ. Engr., Austin & Ashley, Los Angeles, Calif.
† Received by the Secretary, May 22, 1925.
‡ "Studies in Engineering No. 1", Univ. of Minnesota.

found values in that series. By this process, the third substitution will almost invariably give values sufficiently precise.

Extreme precision in secondary stress determination is not only unwarranted by reason of failure to secure the basic assumptions of analysis, but is also rendered futile by the modifying effect of certain factors which are not considered in the usual analyses. Under the first heading come such considerations as:

(1) The assumption of perfect fixity at the truss intersections;

(2) The assumption of perfect elastic deformation of the members between the intersections; and

(3) The assumption of constant moment of inertia.

According to the elastic theory there is bound to be considerable variation in the fixity because the moment of inertia of some gussets may be as great as that of the truss member itself, whereas the moment of inertia of other gussets may be only a small fraction of that of the main member. Indeed, German and Swiss tests, as well as tests at the University of Illinois, have indicated the lack of perfect fixity in gusset-plate connections. This lack of fixity is due to (*a*) rivet slip and (*b*) elastic deformation. The variations from the assumption of perfect fixity have been characterized* as "not introducing serious errors into the results". This finding is to be expected, but it would indicate the uselessness of highly refined secondary stress calculations.

Due to the variant moment of inertia and length of the gussets, there is a variant and appreciable zone of restraint past the intersections, with a resultant effect on the secondary stress.

The assumption of constant moment of inertia is, of course, never realized in practice. Deep and heavy gussets and splice members will modify materially the moment of inertia of the section and, consequently, the secondary stresses calculated on the premise mentioned.

In his discussion† D. B. Steinman, M. Am. Soc. C. E., mentions a variation in the usual secondary stress procedure which involves the recognition of a variable moment of inertia in the members. This procedure should largely compensate for the two latter considerations.

The total stresses, primary plus secondary, as determined by the usual methods, are somewhat modified by the existence of certain concomitant conditions not usually analyzed or considered. The more obvious of these are:

(1) The effect on the unit primary axial stress of increased section due to gussets and splice members.

(2) The effect on the unit primary flexural stress of increased moment of inertia due to gussets and splice members.

(3) Faulty fabrication, including such items as incorrect rivet spacing, inaccurate facing of compression surfaces, sections that are in wind, incorrect shop lengths, etc.

(4) Initial stress in sections due to the effect of the straightening rolls.

(5) Temperature variation between different members of the truss.

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* *Bulletin No. 104*, Univ. of Illinois.

† *Proceedings*, Am. Soc. C. E., February, 1925, Papers and Discussions, p. 297.

(6) Eccentricity produced by flexure in the member between the panel points.

(7) Bending in the verticals caused by deflection of floor-beams.

(8) Torsion in chord members due to deflection of the floor-beams extending over part of the span.

(9) Longitudinal stress caused by friction of roller bearings at a movable reaction.

(10) Decrease in axial stress due to the fact that the floor system assumes load.

(11) Non-uniform stress intensity in outstanding legs of angles, splice members, etc.

These various factors produce stress variations which, in general, are small individually. The design should be arranged so as to correct for them as far as possible, but the calculation of their precise values is commonly disregarded by engineers. This procedure is justified by reason, for their complex nature and erratic occurrence preclude the possibility of a precise analysis without an expenditure of time out of all proportion to the small additional accuracy attained.

These considerations would indicate the futility of striving after great precision in secondary stress calculations through the medium of rigorous mathematical analyses following the so-called "precise methods". As mentioned, Mohr's semi-graphical method with the solution of equations by successive substitutions provides a lucid and expeditious analysis, and one which possesses ample accuracy considering the modifying effect of the indeterminate factors alluded to.

Apropos of the moot questions of relative fixity of members at panel points, some recent German strain-gauge tests may be of interest.* They were taken on the center bottom chord gusset and connecting members in a truss 6.0 m. in span and 1.5 m. in height. The truss was loaded at its center upper chord panel point with a load of 50 metric tons. In every instance, a relatively close agreement was found between the actual stresses and those premised from the assumption of fixed ends. The results of this test, as of other similar tests, emphasize the statement that the fundamental theory of fixity of ends as applied to secondary stress analysis is sound; but that highly refined methods of calculation are unwarranted, because of inevitable and indeterminate modifying influences.

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* *Schweizerische Bauzeitung*, September 23, 1923.

FLOOD FLOW CHARACTERISTICS
Discussion*

By Messrs. H. V. Hinckley, Hardy Cross, E. W. Lane, Gerard H. Matthes,
Ford Kurtz, Charles W. Comstock, Thaddeus Merriman, Weston E.
Fuller and H. de B. Parsons.

H. V. Hinckley,[†] M. Am. Soc. C. E. (by letter).[‡]—For some time the
writer has had in mind advancing a formula similar to Equation (2),[§] and
of the form, $Q = 5\,000 \sqrt{M}$. He realizes that the constant, 5 000, might
have to be enlarged to apply to possibly forty localities. The use of the symbol,
Q, is somewhat objectionable, as no quantity is to be expressed, only a factor
in the quantity.

Since the writer has studied the paper (which is both timely and inter-
esting) and, in connection with it, the previous contributions on flood flow
by Weston E. Fuller, M. Am. Soc. C. E., and the late Emil Kuichling, M.
Am. Soc. C. E., it seems to him unwise to propose definitely the constant, 5 000,
at this time because the author's Table 2,[||] covering more than 1 000 records,
shows 57 cases where the factor, 5 000, is materially exceeded, of which 23
cases exceeded even 10 000. In studying these data, to offset the inadequate
length of observations, a factor was applied to the readings multiplying them
by quantities between 1 and 3.40, the member, 3.40, applying to records of
one year only or of indefinite and unknown periods. This is in accordance
with the recommendation in Fuller's paper.[¶]

For example, Table 6 shows 6 of the 23 cases, in which the author's value
of C (10 000) would be replaced by numbers from 21 400 to 37 400. The mul-
tiplier, 3.40, was used in each case. It may yet be possible to eliminate dis-
crepancies and make use of the formula, $Q = 10\,000 \sqrt{M}$.

The writer believes that the first essential is to have the constant, C, large
enough to cover all the absolutely maximum cases, no matter how large it
may need to be. Then, as suggested by the author, definite percentages might
be adopted to represent certain drainage basins—if it is possible to tell the
relation of future floods in different parts of the continent or of the globe.

It seems hardly necessary to mention that an average of the flood flows
recorded would be far less than the average actual maximum—if there were
such a thing—as fully one-half of the values given as maximum floods are
not maximum at all except possibly for a day.

TABLE 6.—COMPUTATION OF COEFFICIENT, *C*.

Area, in square miles.	Years of record.	Run-off, in second-feet per square mile.	Multiplier.†	Maximum run-off expected, in second-feet per square mile.	$\sqrt{M}$	*C*.
20*	Indefinite	1 800	3.40	122 000	4.47	27 400
22*	"	1 341	"	100 000	4.69	21 400
44*	"	1 363	"	204 000	6.63	30 800
143†	"	600±	"	292 000	11.96	24 400
544†	"	432	"	818 000	28.32	35 000
831*	"	382	"	1 079 000	28.83	37 400

* Data taken from Table 37 of the paper on "Flood Flows", by Weston E. Fuller, M. Am. Soc. C. E., *Transactions*, Vol. LXXVII (1914), p. 650.

† Discussion on "Flood Problems", by H. P. Eddy, M. Am. Soc. C. E., *Transactions*, Vol. LXXXV (1922), Fig. 45, p. 1526, and Table 5, p. 1528.

On Fig. 11, Line *C,* by the writer, from studies made for the State of Oklahoma, compared with the lines, *M* and *K* (all three lines being for maximum second-feet per square mile), shows how the maximum recorded floods have become greater as the records have increased the working data.
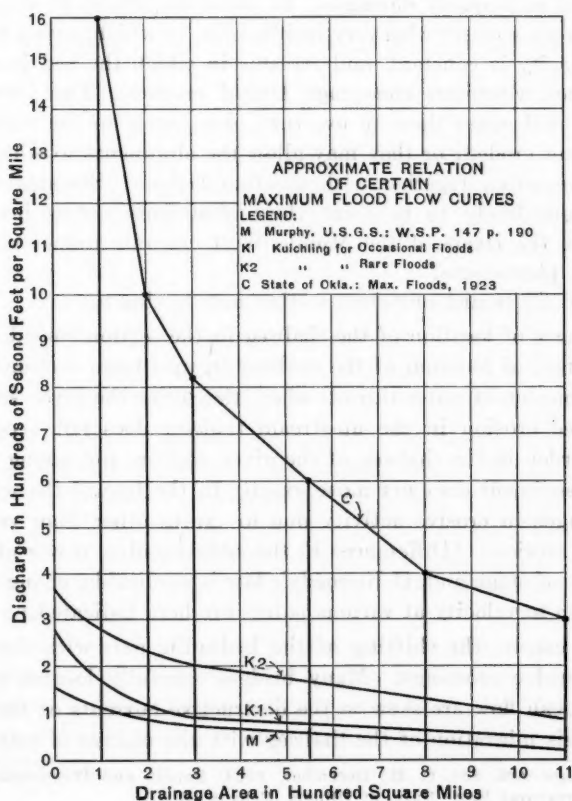

APPROXIMATE RELATION
OF CERTAIN
MAXIMUM FLOOD FLOW CURVES
LEGEND:
M Murphy. U.S.G.S.: W.S.P. 147 p. 190
K1 Kulchling for Occasional Floods
K2   "   "   Rare Floods
C State of Okla.: Max. Floods, 1923

Fɪɢ. 11.

The author seems to have his flood values under good control. Nos. 275 and 493 (Table 2), however, apparently give values for the constant, C, of slightly more than 10 000, being 11 683 and 10 029, respectively.

It would be interesting to know whether the author finds that any of the drainage basins named in Plate VIII* are found to have definite values or percentages of the constant, $C = 10\,000$; for example, whether the flood values for any one drainage district are within the limits of the 10% class and have maximum values less than $Q = 1\,000\ \sqrt{M}$.

HARDY CROSS,† M. AM. SOC. C. E. (by letter).‡—By listing "certain data and considerations that are often left unrelated", the author has prepared a most interesting paper. The whole subject is in a chaotic state, and discussion of it necessarily depends much on individual opinion. Any effort to clarify, systematize, or in any way make definite the problems involved, is a professional and public service. The next fifty years will see much development along the line of placing river hydraulics on a scientific basis.

The special problem of provision for flood flow at river crossings is commonly one of erosion phenomena and only occasionally or secondarily one of back-flow and consequent damages. It seems important to emphasize a distinction between streams with very mobile beds, in which, owing to bed erosion, the flood velocity is constant, and streams in which the bed is so stable that increased flood velocities encourage lateral erosion. The former undercut bridge piers and cause them to overturn, due partly to the unbalanced pressure of silt not eroded; or they may allow the abutments to slide into the bed of the stream, often together with the foundations. Streams of the second class are more likely to back-cut their abutments. Any bridge engineer familiar with the rivers of the Middle West can cite numerous examples of both types of phenomena.

Scour and fill in any one cross-section may be due to:

(a) Changes of location of the thalweg in the section considered.

(b) Changes of location of the thalweg in up-stream sections resulting in a change of burden of water threads where they cross the given section. (Note that increased erosion in the up-stream thalweg does not necessarily mean increased burden in the thalweg of the given section, for, owing to its greater inertia, the sediment does not move exactly in the line of the water threads.)

(c) Changes in erosive activity due to causes other than velocity at any point in the section. (Differences in the eddy-forming power of obstructions with changes of stage and, if Kennedy's law is applicable, changes in the relations of depth to velocity at various points are here indicated.)

Of these causes, the shifting of the hydraulic axis with the stage is important at bridge crossings. Many bridges carefully located square to the thalweg of mean flow are skew to the destructive currents of the falling flood stage; but this migration of the thalweg with the change of stage is only one

of the causes of increased erosion on a falling stage. Water-soaked banks are another important element. Most serious in many cases is the tendency to flank the abutment by a back-cutting eddy below the approach fill.

It is important also to distinguish between the action of protracted and that of "flashy" floods. The latter, of course, have not time to improve the hydraulic efficiency of the bridge opening and, therefore, back-water may produce considerable hydrostatic heads against the approaches.

As bearing out and supplementing certain statements of the author with reference to erosion in silt-bearing streams, some selected conclusions of the twenty general rules formulated in the case of the Indus by the Indus River Commission* are worth quoting:

"The progress of erosion at any point is usually determined by the condition of the river for some miles up stream.

"Erosion is most severe when a bank, which consists of clay or heavy soil at the top, is underlain by sandy strata which can be readily undermined."

*   *   *   *   *   *   *   *   *   *

"Erosion is generally more active during a falling than during a rising stage of the river, as in the former case the unsubmerged and overhanging bank is charged with moisture which weights it and diminishes its cohesion."

*   *   *   *   *   *   *   *   *   *

"An embayment is, almost invariably, formed first below this point [a resisting projection of the bank] owing to the swirl of the water produced by the construction.

"If this embayment becomes sufficiently deep, the projecting bank will be outflanked and washed away."

Although it is of quite as much economic importance as any other phase of bridge design, the problem of the design of river crossings, as distinguished from the structural design of bridges, is one on which little accurate information is available. It is encouraging to note how fully the author and his co-workers realize the intricacy of the problem. It is to be hoped that such papers as this may bring out the huge mass of experience on this subject possessed by drainage engineers of large railways. The writer believes that the subject can be put on a basis approximately as scientific as has been reached in recent years in the design of spillways.

E. W. LANE,† ASSOC. M. AM. SOC. C. E. (by letter).‡—This paper is valuable in that it brings together a large amount of data on flood flows. This information may serve as a rough guide in the design of hydraulic structures, but the writer is of the opinion that a much more reliable indication of probable floods may be obtained, particularly for those in streams for which a few years' discharge records are available, by a different method from that used by the author.

The discharge rate of any flood is the product of innumerable factors, and the combinations of these factors which cause floods on any stream give rise to discharges which have wide ranges of magnitude. One of the important factors is the area of the water-shed, and this is the only one which

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* Indus River Commission Record, Sind, P. W. D., Part IV, Conclusions.
† Salisbury, Conn.
‡ Received by the Secretary, April 11, 1925.

the author includes in the formula which he adopts. After its effect is elim-
inated as far as possible, the data given still show a range of discharge repre-
sented by numerical coefficients of from 50 to 10 000. When the selection
of the proper coefficient is left to the judgment of the individual engineer,
it is obvious that the formula itself is of comparatively little assistance.

Although many factors combine in causing floods, they may be divided
into two groups: (1) Those depending on the physiography of the basin in
question; and (2) those depending on the weather. In the first group, the
most important factors are the area, topography, geology, shape, and permea-
bility of the water-shed, and the availability of storage in lakes and swamps.
Those of most importance in the second group are the precipitation, tem-
perature, and the snow and ice conditions on the water-shed. Vegetable
cover and the condition of storage of water in soil, lakes, and swamps are
really products of factors from each of the two groups.

For a given point on a stream, the factors in the first of these groups are
constant; that is, these conditions remain the same. Weather conditions are
proverbially variable, but, although they cover a wide range, they follow some
probability law. This is particularly true of the weather phenomena that
give rise to floods.

As a rational flood formula must take into account both these classes of
influences, it should be made up of two parts. One would be a factor express-
ing the physical characteristics of the stream; this would be a constant for a
given point, but would vary somewhat at different points on the same stream,
still more for the streams of a given region, and widely for the streams of
a continent. The other part would express a probability law which would
be practically the same over a comparatively large area, but not necessarily
the same for all water-sheds.

The writer believes that any formula which does not include the effect
of both these groups of influences would be only a rough approximation and
of little usefulness. Most formulas have a factor for water-shed area, some
consider to a limited extent the physical characteristics of the drainage basin,
and a few have some indication of the time element; but the Fuller formula*
seems to be the only one that gives numerical values to the probability factor
and also considers the basin characteristics. As an example of the necessity
of considering the water-shed characteristics may be cited the comparison
of the Deerfield River at Charlemont, Mass., with Miller's River at Erving,
Mass.; these streams have nearly the same drainage area, but the former
gives flood discharges five or six times as great as the latter. The Souhegan
River at Merrimack, N. H., gives discharges about three times as large as
the Swift River, at West Ware, Mass., although their drainage areas are
nearly equal. If as great variations as these occur in as small a region as
New England, where weather conditions are so nearly uniform, how much
larger variations can be expected when the whole United States or the entire
world is considered?

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* *Transactions*, Am. Soc. C. E., Vol. LXXVII, (1914), p. 567.

The author has objected to the time element in flood formulas as "too often the basis of error". It is true that it may sometimes be misunderstood by those who blindly apply formulas without much thought as to their meaning; but must formulas be made so simple that the least intelligent or informed may readily understand them, thus denying a useful tool to those who are capable of using it? Progress is not made by that method. As a matter of fact, progress in engineering science is so rapid that the products of the best minds in one decade are used by all the fairly well informed in the next, so that even if a development seems complicated at the time it is made, in a few years it will be considered relatively simple.

The writer believes that the time element in a rational flood formula is a simple thing, well within the comprehension of an ordinary mind. Unfortunately, its meaning has been needlessly obscured in the development of the Fuller formula. Suppose there was a long record (say, 1 000 years) of the flood discharges of a stream at a given point, arranged in order of their magnitudes, beginning with the largest. Suppose the one-hundredth in the list had a discharge of 5 000 sec-ft. There have been 100 floods in the 1 000 years with a discharge of 5 000 sec-ft., or more. Therefore, the average interval between floods of 5 000 sec-ft., or more, is $\dfrac{1\,000}{10} = 10$ years. If a spillway on this stream at this point had a capacity of 4 999 sec-ft., its capacity would be exceeded once in 10 years on the average. The probability of its capacity being exceeded in any one year would be 0.10. For convenience, call the average interval, in years, I. Then the probability of occurrence in any one year would be $\dfrac{1}{I}$. The interval used by Mr. Fuller, in which the chances of a flood of that magnitude occurring are equal to the chances that it will not occur, is, therefore, $\dfrac{I}{2}$. To use the example of the spillway again, the interval used by Mr. Fuller, which he computes from his average maximum flood or median maximum flood, is the interval for which the chance of the spillway capacity being exceeded is equal to the chance that it will not be exceeded—one chance out of two, or 0.5. Since the chance in any one year is 0.10, the period for which the chance would be 0.5 would be five years, or $\dfrac{I}{2}$. The writer believes that the use of the interval, I, is more easily understood than the interval used by Mr. Fuller.

This discussion may seem elementary, but it indicates how simple the element of time in a rational flood formula really is. Some doubt as to the reliability of the probability formula is apt to arise in the mind of the engineer when he considers a case where several unusually large floods come within a short interval, or where severe flood conditions occur simultaneously over a wide region, as happened in March, 1913. He should remember that the theory on which life insurance is based is not disproved because several members of the same family may die in the same year, or because an epidemic

of influenza sweeps over the country, claiming the lives of thousands. These cases in the field of insurance are paralleled in the field of flood probability.

To try out the writer's ideas on the development of a logical flood formula, a study was made of the floods of New England rivers from the data compiled by C. H. Pierce, M. Am. Soc. C. E.* The probability or average interval curve was computed for each station having sufficiently complete records. The flood discharges at the station in question were arranged in order of magnitude, beginning with the largest, and numbered consecutively, the largest being No. 1. Disregarding the error due to the fact that the record covers only a short period, the average interval, I, between floods as large or larger than any one of those listed, can be found by dividing the length of the record at that station, in years, by the serial number of the flood. Thus, with a 10-year record, the value of I for the magnitude of Flood No. 4 would be $\frac{10}{4} = 2.5$ years. Average interval curves were then drawn showing the relation between the flood magnitude, in second-feet per square mile, and the average interval, I. Logarithmic, semi-logarithmic, and probability paper were used, but the best results were obtained from the semi-logarithmic paper.

In Fig. 12 are shown the resulting curves. Many of the data were in the form of peak flows, but other records consisted of one reading per day, the higher of two readings per day, and the average flow, the various kinds being indicated by different symbols. On account of the wide difference in unit flows, it was necessary to divide the data into two parts, and use different scales, in order to plot them on a diagram of reasonable size. Table 8 gives the principal data regarding the stations the records of which were used. On account of the fact that the length of record was insufficient to determine accurately the average interval for floods of greater magnitude, the values indicated by the highest points are made less prominent. It will be seen that all the curves, as indicated by the data, approach reasonably close to straight lines, and converge to approximately the same point.

Fig. 12 shows that the average interval between floods of various magnitudes for the New England rivers can be expressed by an equation of the form:

$$Q = K \, (\log I + B) \dots\dots\dots\dots\dots\dots (4)$$

in which, Q is the discharge in second-feet per square mile which would be equalled or exceeded on the average once in I years; K is a constant for the station in question; and B is a constant for New England streams and probably nearly constant for a much larger area.

It will be seen that this formula is made up of the two parts previously mentioned as being what one would expect in a rational flood formula, namely, a constant, K, representing the physical characteristics of the water-shed, and a probability factor, $\log I + B$, representing the weather variability.

Although the writer had available only the data for New England streams, from the results of Mr. Fuller's studies, he believes that an expression of

* "Flood Flows of New England Rivers", *Journal*, Boston Soc. of Civ. Engrs., October, 1924, Vol. XI, No. 8.

FLOOD PROBABILITY CURVES
FOR NEW ENGLAND RIVERS
For Data of Stations 1 to 18 see Table 8
SYMBOLS

Peak Flows.
Average for 24 Hours
One Reading a Day
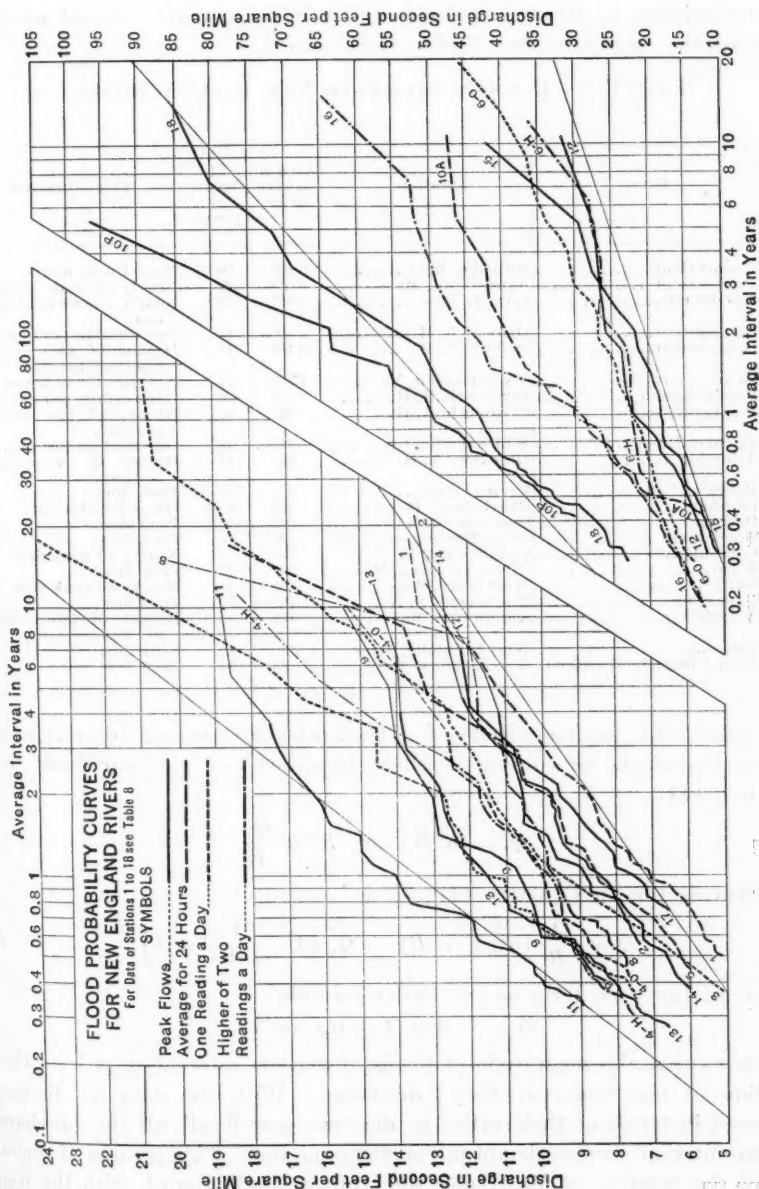Higher of Two
Readings a Day

Fig. 12.

the form of Equation (4) would apply to nearly all the rivers in the United States east of the 95th Meridian, which is roughly the western boundary of the States immediately west of the Mississippi River. He is inclined to doubt its applicability to the "plains" rivers, but believes that it would apply to many of the mountain and Pacific Coast rivers.

TABLE 8.—RUN-OFF STUDIES OF NEW ENGLAND RIVERS.

No.	River.	Station.	Area, in square miles.	Length of record, in years.	Type of record.
1....	Connecticut..............	Holyoke, Mass	8 390	20	Mean for 24 hours.
2....	Connecticut..............	Sunderland, Mass.......	8 000	21	Nearly all peak flows.
3....	Merrimack	Lowell, Mass.....	4 097	72	Nearly all one reading per day.
4–O.	Connecticut..............	Orford, N. H............	3 100	10	One reading per day.
4–H.	Connecticut..............	Orford, N. H............	3 100	11	Higher of two or more readings per day.
5....	Saco.....................	West Buxton, Me.......	1 500	17	Average for 24 hours.
6–O.	Pemigewasset.............	Plymouth, N. H.........	615	26½	One reading per day.
6–H.	Pemigewasset.............	Plymouth, N. H.........	615	12	Higher of two readings per day.
7....	Machias River	Whitneyville, Me.......	465	18	One reading per day.
8....	Ashuelot	Hinsdale, N. H..........	440	15	Higher of two readings per day.
9....	Miller's..................	Erving, Mass...........	372	10	Peak flow.
10–A.	Deerfield	Charlemont, Mass.......	332	10½	Mean for 24 hours.
10–P.	Deerfield	Charlemont, Mass.......	332	10½	Peak flow.
11....	Housatonic................	Great Barrington, Mass.	280	11	Peak flow.
12....	Lamoille..................	Cady's Falls, Vt.........	280	10	Nearly all peak flow.
13....	Ware	Gibb's Crossing, Mass...	201	12	Peak flow.
14....	Swift	West Ware, Mass.......	186	14	Nearly all peak flow.
15....	Souhegan.................	Merrimack, N. H.......	168	10	Peak flow.
16....	Westfield.................	Knightville, Mass	162	15	Higher of two readings per day.
17....	Quaboag..................	West Bumfield, Mass ...	150	12	Peak flow.
18....	Middle Branch, Westfield.	Goss Heights, Mass.....	53	14	Peak flow.

Representing the magnitude of a flood with an average interval of one year as Q_1, which, for convenience, may be called the "one-year flood", from Equation (4):

$$Q_1 = K B, \quad \text{or} \quad K = \frac{Q_1}{B}$$

Substituting this in Equation (4):

$$Q = \frac{Q_1}{B} (\log I + B) = Q_1 \left(1 + \frac{1}{B} \log I\right) \dots\dots\dots\dots (5)$$

This is of the same form as the Fuller formula:

$$Q = C A^{0.8} (1 + 0.8 \log T)$$

and shows that the magnitude of floods of various intervals is a logarithmic function of the "one-year flood" discharge. With the data on discharge expressed in terms of their ratios to the "one-year flood" all the "discharge-average interval" curves should fall on the same line. This permits of combining all the records, as Mr. Fuller did, into a single record, with the result shown in Fig. 13.

In determining this curve, all the different kinds of records were used, as there seems to be no great variation between the shapes of the curves from

different kinds of observations. It is possible, however, that if enough records were available, a difference might be shown.
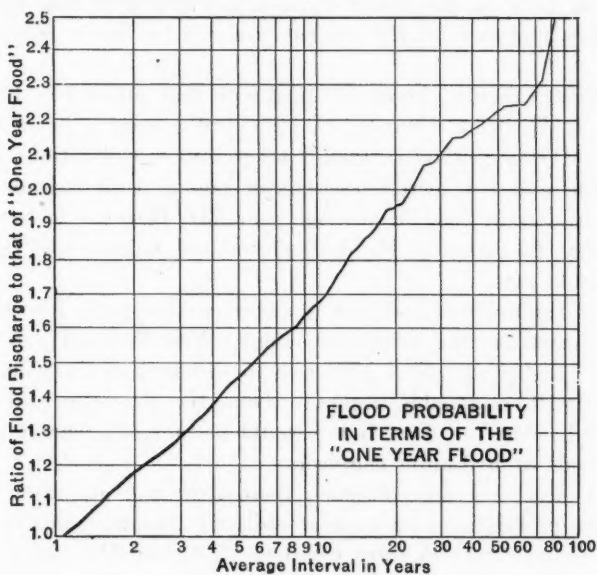
Fig. 13.

The value of $\dfrac{1}{B}$ was found to be 0.690, which differs somewhat from the 0.80 found by Mr. Fuller. It is possible that if only the "average for 24 hours" and "one reading a day" records were used, a better agreement with the Fuller formula would have resulted. Mr. Fuller's data were based on records supposed to represent the average for 24 hours. In a great many cases, however, the observation was made once daily, which reading was assumed to give the mean flow for the day.

The writer does not claim originality for his method of determining flood probabilities, or for the formula, although they were independently developed to find later that some points were previously used by others. He has not hesitated to borrow from others when their methods seemed sound. He does believe, however, that the method he has outlined leads to a flood formula which combines rationality, simplicity, and accuracy to as great an extent as is possible for so complex a phenomenon as floods.

The method here given for working out probability curves and deducing the flood probability formula has certain advantages over the methods heretofore proposed. Although the form of the result is very similar to that reached by Mr. Fuller, the method is less laborious, and has the great advantage that the meaning of the result is definite and readily understandable, as compared with Mr. Fuller's "average maximum flood" and "median maximum flood", which are very difficult to understand, "if not indeed practically meaningless", to use the words of a well-known hydraulic engineer. By the

writer's method, the larger floods, the average intervals of which are less reliably determined, are automatically eliminated in the determination of the one-year flood, while in the Fuller method, a single flood considerably larger than the other members in the record exercises too much influence on the results.

The writer's method is almost identical with that outlined by R. E. Horton, M. Am. Soc. C. E., in his discussion* of Mr. Fuller's paper, but is an improvement on the Horton method in that it takes into consideration all the floods of the stream, and not merely the largest one in each calendar year. In using only the largest flood in any one year, Mr. Horton's method is subject to the criticism he makes of Mr. Fuller's method, namely, the failure to use all the available data.

If time and data were available to carry out the work for other sections of the United States as the writer has done for New England, and a study was made of the conditions which give rise to different index values, in order to determine with reasonable accuracy, from the character and location of the water-shed, what the flood index would be, a tool of great value would be in the hands of hydraulic engineers.

The work necessary for collecting these data and analyzing them is so great as to be prohibitive to a man of ordinary resources, but its value to a single piece of construction work could easily exceed the cost of the entire investigation. The logical body to make such a study is the Society's Special Committee on Flood-Protection Data, and the request of this Committee that the Society appropriate funds for an investigation of flood flows should be granted. The results would add many times the cost of the work to the wealth of the nation.

For the last ten years, the writer's work has been on flood prevention; before starting the studies outlined in this discussion, he was quite skeptical of flood flow formulas. Working entirely independently, he has arrived at the same general form of equation that was reached by Mr. Fuller, although the meaning of the terms of the two formulas, and the method of deriving them, are somewhat different. He is convinced, therefore, of the correctness of the general formula and believes that further study should be along the line of determining the constants for various streams, and the limit of applicability of the formula.

Some objection has been made to the logarithmic form because it fails to give a maximum value which can never be exceeded. The writer does not consider this a serious objection. It is based on the assumption that there is a maximum flood, but this has never been proved. With the logarithmic form discharges only a few times as great as the 10-year magnitude become so infrequent as to be almost geological periods apart, in which time the river valley may become a sea bottom. It is much better to have an empirical formula agreeing with the data within practical limits than one which is logical according to an improved theory, but may not agree so well. With equally good agreement with the flood data, the writer would prefer a simple

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logarithmic formula to one having a maximum limit and being thereby complicated.

In this discussion it is not the writer's intention to develop a satisfactory flood discharge formula, for much study is necessary before such a result can be reached, but he desires to point out some of the basic requirements of a satisfactory formula, the lines along which study should be made, and some of the methods that he believes most likely to lead to the desired result.

One of the points requiring further study is the influence of area on discharge. Mr. Jarvis has adopted the area factor as the 0.50 power, Mr. Fuller derives the 0.80 power from nearly equally good data. Talbot used the 0.75 power, and other results have been found by other investigators. It is doubtful whether any single figure will cover all cases, and where an attempt is made to predict flood discharges at one point on a river from the data at another point having a considerably different drainage area, or different water-shed conditions, an exercise of judgment would be necessary.

For example, at Dayton, Ohio, two rivers of roughly the same drainage area as the Miami River join it at nearly the same point. As all the areas are small enough to be covered in many cases by the same storms, the flood discharges of the Miami below the junction, per unit of area drained, is only slightly less than that of the individual tributaries, although the area drained is about three times as large. As an example of another condition, the case of the Arkansas River might be cited. Its source is in the mountains, but the lower portion flows through the plains. The flood characteristics of the mountainous parts of the drainage area are entirely different from those below. It is evident that any area factor would have to be applied with judgment, but the writer believes that when flood probabilities are worked out for many stations on the same rivers, it will be possible to make quite close estimates of the area factor for ordinary cases.

The most important feature of a flood is usually the peak flow, but the published records are generally in the form of the average flow for the day, and give peak flows only when they are very large. For this reason, in computing flood probabilities, it is usually necessary to work out probabilities of average daily flows, and from this to determine the probability of peak discharges. A great deal of further study is necessary before this relation can be satisfactorily established. Mr. Fuller has derived an expression for the relation between average daily flow and peak flow, but this expression is one of the greatest objections to the Fuller formula. With great care he finds the probability of average daily flows for all parts of the United States, and to obtain the result nearly always desired—the peak flow—he multiplies this by a factor based on only thirty-three observations, many of which differ 100% from the values computed from the formula adopted. Data on the ratio of peak to average daily flow are scarce, and probably Mr. Fuller used all that were available.

His assumption that this ratio is only a function of the drainage area leaves out an important consideration and probably accounts for much of the variation between his formula and the actual results. For instance, the Hud-

son River, at Mechanicsville, N. Y., and the Oswego River, at Battle Island, N. Y., have nearly the same drainage area, but a glance at their hydrographs* will show that the ratio of the peak flow to the average 24-hour flow for these rivers would differ widely. For the region east of the 95th Meridian, the writer believes that an expression combining the drainage area and the value of Q_1, which is really an index of the variability of the flow, would give much more satisfactory results.

As an example of the error resulting in some cases from the Fuller formula might be cited the Arkansas River at Pueblo, Colo., with a drainage area of 4 600 sq. miles. The average ratio for twenty-two peaks was 2.5, which agrees well with that cited by the author for the great flood of 1921, while the ratio of the Fuller formula for a stream having this area would be 1.159. In one case the discharge rose to 12 500 sec-ft., dropped to 2 100 sec-ft., and rose to 10 800 sec-ft. in a single day. All these floods were due to cloudbursts on a small part of the drainage area, while there may have been no evidence of a storm at other points.

If a great many records were available, by plotting a curve like that on Fig. 13, using peak-flow data only, and another using the averages for the 24-hour data only, some clue could be obtained regarding the ratio, but much more data like those shown on Fig. 12 for the Deerfield River, giving both the peak flow and the average daily flow for a long period of record, will be necessary before a satisfactory result will be obtained.

Although the application of the rational type of formula is not as easy a matter as that of the type suggested by the author, it is really a rather simple process. In general, there are three conditions under which predictions of future floods are desired. The point may be (1) on a stream near a station having a long discharge record; (2) on a stream near a station having a short-period record; or, (3) on a stream remote from a discharge station or having no measuring station.

The most accurate predictions can be made from the conditions in Class (1), but even here the general formula is of material assistance. For example, an attempt was made to develop a flood probability curve for the Miami River, at Dayton, from a long-period record there. It was found that the data on the larger floods were so limited that formulas giving widely different probabilities for large floods fitted the data equally well. By means of the general formula, a much more accurate flood probability could have been determined.

The number of flood records is rapidly increasing and will continue to increase. This is not only beneficial to a determination of an accurate formula, but also to the application of it. At present, some form of discharge records exist at one or more points on most of the large and medium sized streams of the United States, and reasonably reliable predictions will be possible for them when a thorough study has been made to determine the best prediction methods. Most of these fall in Class (2). The flood probability for the point in question can be determined by examination and com-

* "The Economics of Hydro-Electric Development," by Daniel W. Mead, M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., April, 1924, p. 426 (Fig. 3).

parison of the probabilities as determined for adjacent measuring stations. This can best be done by drawing the probability curves as was done for Fig. 12, and determining the "one-year flood" from the straight line most closely fitting the points, but an approximate method may be used without recourse to plotting. In this approximate method, the floods are arranged in order of magnitude, beginning with the largest, and numbered. The one having the number equal to the length of the record may be considered as the "one-year flood". Substituting this in the formula,

$$Q = Q_1 \left(1 + \frac{1}{B} \log I\right)$$

gives the probability of floods of other magnitudes and intervals.

To investigate the extent of the error which might occur in the probability curves thus determined, a study was made by dividing the 72-year record of the Merrimack River, at Lawrence, Mass., into periods of various lengths, and comparing the "one-year floods" determined from these periods with that determined from the 72-year period. The result, as shown in Fig. 14, is highly satisfactory. For example, from 5-year periods, more than 75% of the determinations were within 20% of the proper value, and with 10-year periods about 98% were within 20% and about 75% were within 10%, while with 20-year periods about 98% were within 10 per cent.

PROBABLE ACCURACY OF
APPROXIMATE METHOD
OF DETERMINING THE
"ONE YEAR FLOOD"
From Investagation of the
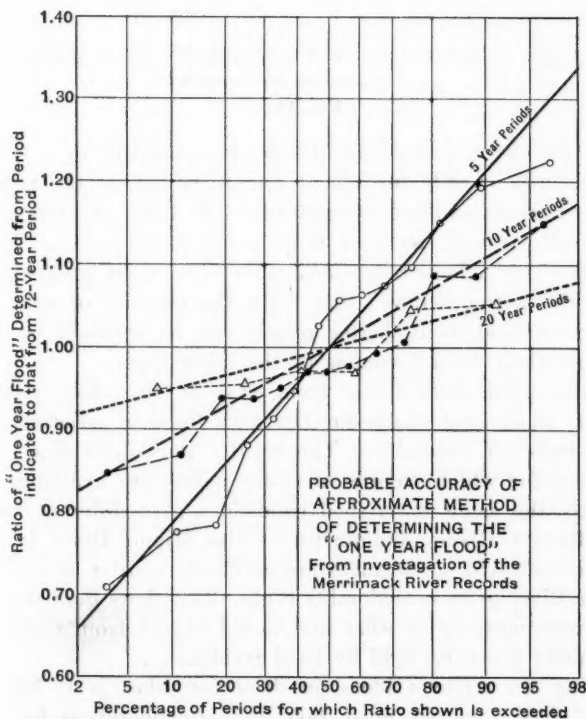Merrimack River Records

FIG. 14.

Fig. 15 was prepared to show how closely the probability curves, as determined from different parts of the record, agreed with each other. For this purpose, the 72-year record of the Merrimack was divided into two 36-year periods and four 18-year periods and the curves for all the periods plotted together. The result shows a very satisfactory agreement.
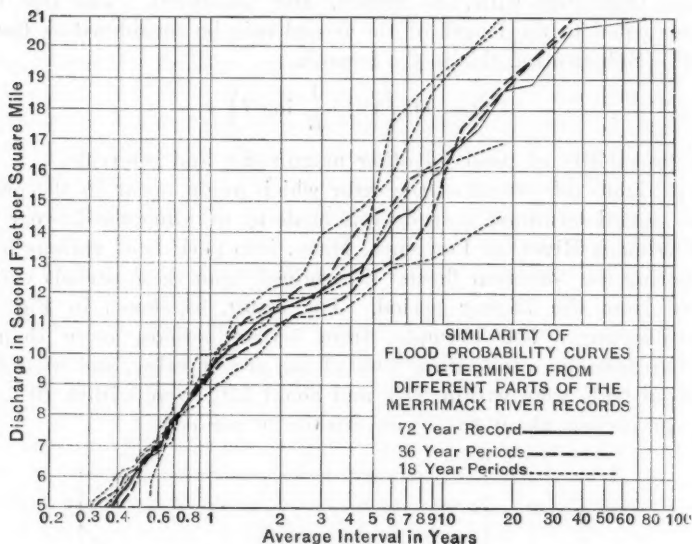

SIMILARITY OF
FLOOD PROBABILITY CURVES
DETERMINED FROM
DIFFERENT PARTS OF THE
MERRIMACK RIVER RECORDS

72 Year Record————
36 Year Periods— — — —
18 Year Periods..........

FIG. 15.

Class (3), the determination of the flood probability of a stream which has no flow records or only records at remote points, is the case most frequently met by bridge and highway engineers. As there are comparatively few records for small streams, most of them come in this class. In this field, formulas of the type adopted by the author have their greatest usefulness. Nevertheless, the writer believes that, with the exercise of care and a reasonable amount of judgment, better results can be obtained by the method outlined by him than from formulas of the other type.

To determine flood probabilities from cases in this class, it is necessary to compare the water-shed characteristics with those of adjacent water-sheds for which records are available. The results thus determined are not so reliable as those for which records at closer points are available, as streams with apparently the same water-shed conditions give different results. The Mad and Stillwater Rivers, tributaries of the Miami River in Ohio, have basins which in outward appearances are as nearly similar as could be found. The Stillwater River gives considerably higher flood flows than the Mad River, which is entirely contrary to what one would expect from their names and from the opinions generally held by local residents.

Summarizing the writer's ideas on flood formulas, it is his belief that a satisfactory flood formula must take into consideration both the basin characteristics and the time factor, and that, although considerable progress

already has been made in the development of such a formula by Mr. Fuller, great improvements are still possible, but, as these require time and expense beyond the resources of a single man, this study should be carried on by the Special Committee on Flood-Protection Data. The writer heartily agrees with the author as to the need for this type of study and believes that the Society should provide the Committee with the necessary funds to make an adequate investigation possible.

GERARD H. MATTHES,* M. AM. SOC. C. E.—This noteworthy paper clearly represents the fruits of considerable labor on the part of the author in compiling data for use in proportioning waterway areas for bridges and culverts. The treatment of the subject appears to be principally from that angle, which accounts for the fact that the author is concerned mainly with maximum discharge rates, as shown in Table 2.†

Of practical interest is the following specification of the U. S. Bureau of Public Roads mentioned by the author:‡

"The waterway shall be adequate for the passage of ordinary flood water * * * and be adequate for the passage of extreme flood water unless other provisions are made for same. By ordinary flood water is meant the stage that is likely to occur once in ten years."

This appears to be a sensible basis for designing such structures, and should operate to provide bridges and culverts adapted to local conditions. Thus, in a region like Colorado where cloudbursts are common, the 10-year criterion would provide structures capable of passing extreme flood flows except where they could be by-passed, while in a State like Maine where cloudbursts are of rare occurrence, much smaller waterways will suffice. It would be interesting to know what considerations led to the adoption by the Bureau of the 10-year flood as the criterion for ordinary flood flow, as this might be of value also in the design of other structures built to resist flood waters.

Mr. Jarvis could have made Table 2 of more direct value if in Column (5) he had given figures representing the "ordinary flood water" as here defined.

Two aspects relating to flood studies have suggested themselves to the speaker in reading this paper: First, the imperative need for better means of interpreting the enormous dissimilarity in the flood characteristics of streams; and, second, the desirability of establishing uniform methods of compiling flood data for publication.

As regards the first, Plate VIII§ which is the most complete tabulation of its kind that the speaker has seen, brings out strikingly the wide variation that exists in the maximum flood rates of North American streams, and, by inference, the magnitude of the task that confronts any engineer, not a specialist in flood data, who is called upon to decide what rate of flood run-off to assume in the design of a structure on a given stream. Whether he uses a formula or a diagram, he must exercise a certain amount of judgment in order that the rates of discharge selected shall be commensurate with the character of

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* Cons. Engr., New York, N. Y.
† *Proceedings*, Am. Soc. C. E., December, 1924, Papers and Discussions, pp. 1563–1579.
‡ *Loc. cit.*, p. 1559.
§ *Loc. cit.*, p. 1555.

the stream and the requirements of the particular problem in hand. Methods or devices that make possible the efficient exercise of such judgment is what the Engineering Profession has been seeking for a long time, and this want is, even to-day, far from being filled. The speaker's experience indicates that formulas can be used with safety only by experts who are familiar with the basic data from which they have been derived. This would seem especially true of the modified Myers formula in which the coefficient ranges all the way from 100 to 10 000. This formula fits the data in Plate VIII admirably, but it is not clear how a set of coefficients might be prepared for practical use. Possibly, the author has undertaken such a task, and, if so, it would add materially to the value of his paper if he would supply this information in workable form.

Suitably prepared diagrams are, on the whole, more intelligible than formulas when judgment must be exercised, because they enable the expert and the non-expert alike to visualize many things which a formula will not disclose. For example, a glance will tell what part of a curve is supported by actual observations, and what part has been produced beyond the limits of such data. Again, in dealing with a drainage area for which few or no recorded data are available, the use of a diagram is superior to that of a formula because it permits of more intelligent exercise of judgment. Furthermore, diagrams of the right kind may be kept up to date by entering on them new data as they become available; this cannot be done conveniently in the case of formulas.

The advantages of the diagrammatic method are well illustrated by reference to curves showing the probable frequency of flood discharges of various magnitudes for 173 California streams.* A separate curve is drawn for each gauging station, or section of river, showing the observed data on which it is based, the entire range of floods that may be expected, and the probable frequencies of floods of any given magnitude. For streams on which few or no observations have been made curves are prepared based on other comparable drainage areas for which observed data are available. With such diagrams it is practicable to solve problems relating to the maximum discharge that is likely to occur at a given point in a stated period of years, and direct comparison may be effected between different streams or points on the same stream, while new data may be readily added from time to time or the curves altered in the light of later information. Such diagrams make available basic information the substance of which is open to inspection, and the derivation of each curve is readily seen. Other commendable features are the compactness of the material thus presented, and the uniformity of scale of the diagrams, which permits of showing with reasonable clearness four diagrams on a 6 by 9-in. page.

In giving consideration to the use of formulas and diagrams for publishing flood data the Society's Special Committee on Flood-Protection Data has expressed unanimous preference for diagrams of the general type used in the California report previously mentioned.

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* *Bulletin No. 5*, Report on California Water Resources Investigations.

The author has well said that "there exists an endless variety of conditions that affect flood run-off". Many of these conditions have been made the subject of study, but in the case of large streams two factors do not appear to have gained the recognition that is due them, one being the effect of the geographic arrangement of the tributaries on the building up of the flood wave in the parent stream, and the other the general direction of flow with respect to the movement of great storms. With regard to the first factor, high peak discharges in large rivers are usually traceable to synchronism in the arrival of flood waves from certain major tributaries. Other streams, exposed to storms in all respects similar, may lack this synchronism because of the different configuration of their basins, and their flood crests will be less in height but the duration of flood flow will be correspondingly prolonged.

An interesting illustration of the importance of these factors is afforded by Table 9, showing the greatest flood discharge in a century or more from each of three drainage areas of nearly equal extent. The river stage records, in each case, are of approximately the same length, and the relations between stage and discharge have been so well established as to be subject only to minor corrections.

TABLE 9.—FLOOD DATA FOR THREE COMPARABLE AMERICAN RIVERS.

Stream.	Locality.	Drainage area, in square miles.	Date of maximum flood.	Maximum Rate of Flood Flow :		Coefficient in modified Myers formula.
				Second-feet.	Second-feet per square mile.	
Susquehanna..	Harrisburg, Pa........	24 100	June 2, 1889...	700 000	29.1	4 520
Ohio...........	Wheeling, W. Va......	23 800	Feb. 7, 1884...	494 000	20.8	3 210
Tennessee.....	Chattanooga, Tenn...	21 400	Mar. 11, 1867...	393 000	18.4	2 700

It will be noted that the Susquehanna River at Harrisburg discharges per square mile of drainage area, 58%, and the Ohio River at Wheeling, 13%, more than the Tennessee River at Chattanooga, although the latter drains a basin noted for its heavy storms and having a mean annual discharge greater than those of the others. All three streams are situated in the Eastern United States, two of them, the Susquehanna and Ohio, having their head-waters partly adjacent to each other; all of them drain country in many respects comparable, with mountains at the head-waters partly covered with heavy timber stands; and the climatic conditions do not differ materially except that the Tennessee River Basin lacks the cold winter seasons. Ordinarily, there would be reason to look for a fair degree of similarity in the flood behavior of these three streams. Radical differences exist, however, in the geography of their drainage basins. In the case of the Susquehanna River, two large tributaries, the Juniata and the West Branch, are responsible for the great floods in the main stream. Their basins adjoin and are usually covered by the same storms; their courses are from west to east in the same direction as the movement of storms, and their flood waves crest at about

the same time, the flood peak of one joining that of the other in the channel of the Susquehanna.

On the Ohio River, the conditions are nearly reversed, there being no marked synchronism in the arrival of the main flood crests of the Allegheny and Monongahela Rivers at Pittsburgh, Pa., owing to the peculiar geographical arrangement of their basins and tributaries. In consequence, the maximum flood rates at Pittsburgh and Wheeling are not very high from the standpoint of flood run-off. The flood crests in the Ohio are caused mainly by synchronism in the flood waves of the Kiskiminetas and Youghiogheny Rivers, the water-sheds of which are contiguous and often subject to the same storms. Both streams drain from east to west, contrary to the storm movement; and, before their waters can join, one of them must first empty into the Allegheny and the other into the Monongahela. With an arrangement of this sort maximum flood rates such as occur on the Susquehanna River are physically impossible on the Ohio.

As for the Tennessee, this river and all its main upper tributaries flow in directions opposite to that of storm travel. In addition, there is little synchronism in the discharge of the larger tributaries. Thus, the flood crest of the Hiwassee River passes out first, and that of the French Broad River last.

Obviously, in the case of large streams the synchronism, or the lack of it, is an important factor. On smaller streams, the time of collection and concentration of storm run-off is usually so brief that the geographic arrangement of the tributaries cannot sensibly affect the net result. Thus, in a small basin like that of the Miami River (4 000 sq. miles), the maximum rates of flood discharge caused by the three-day flood of March, 1913, would have been no different if the tributaries, instead of converging in the existing crow-foot arrangement, could have been made to empty into the Miami many miles apart.

To what extent the coefficient of a formula can be made to take account of such divergent geographic conditions is not at all clear. It will be seen from Table 9 that the coefficient in the modified Myers formula is 4 500 for the Susquehanna, 3 200 for the Ohio, and 2 700 for the Tennessee. In the last analysis the coefficient in a flood formula is nothing more than a classification index intended to make possible a comparison between things not strictly comparable. In contrast, the California diagrams treat each stream as a law unto itself and comparisons are avoided. This, to the speaker, appears to be the rational method of dealing with flood phenomena.

The speaker feels that this is an opportune time for emphasizing the desirability of establishing uniform methods of compiling flood data. It is an undeniable fact that, of the existing mass of valuable flood statistics collected to date, only a comparatively small portion is in form for general use. Much of it has appeared in printed form, and some of it remains inaccessible to the profession at large, lying as it does in private files. No uniform system of collating any of it appears to have been advocated.

The demand of the Engineering Profession to-day is for more intensive and extensive compilations than have been made heretofore—more intensive, in the sense that data should receive closer scrutiny and be made to include something more than mere dates and maximum flood rates; more extensive, in the sense that more streams should be studied and existing records be extended back to include information pertaining to early floods. Reviewing the long lists of flood data that have been published from time to time by the Society one is impressed with the wealth of information of this kind that may well serve as a foundation on which to build. The same may be said of the stream flow records of the U. S. Geological Survey. Its early publications give little information regarding floods, but the later ones are quite complete in that respect. Good work is being done also by State bureaus, and here and there much valuable material is available in private offices.

Flood data, to be of maximum serviceable value, must be in such form as to lend themselves to any of the many uses that engineers may wish to make of them. Engineers having dams to build and spillways to design are interested not only in maximum rates, but in the duration of floods and the total volumes of water discharged day by day. The same considerations affect the proportioning of reservoirs for storing flood waters and of detention basins. On the other hand, to the designers of bridges, culverts, flood channels, and levees, maximum rates are of chief interest and knowledge of flood duration or volumes of discharge is not essential. In all these operations, however, one factor stands out in importance above all others, controlling as it does the major aspects of any design, and that is, the probability of the recurrence of floods of various magnitudes. This may well be called the economic factor in any flood problem.

For some time the Special Committee on Flood-Protection Data has been at work devising a simple but comprehensive system to meet these various requirements. This has been referred to in the discussion* by N. C. Grover, M. Am. Soc. C. E., Chairman of that Committee. Attention is here invited to some of the features introduced, in the belief that they go far toward solving the problem of a uniform system for compiling flood data. To facilitate matters and insure uniformity, special blank forms on sheets, 8½ by 11 in., have been printed by the Society, which may be had on application. The headings for these forms are shown on Fig. 16. The complete forms have space for 80 records in Form No. 1; 100 additional records in Form No. 2; 26 records (a to z) in Form No. 3, Table 2; 11 records in Form No. 3, Table 3; and 100 records in Form No. 4. Each form has space at the foot for the reporter's name and the date.

Forms Nos. 1 and 2 show in chronological order all floods that have occurred on a stream during the period covered by continuous records. By "all floods" is meant every rise of which there is record down to and including the least maximum annual flood. The latter, commonly referred to as the "one-year flood", serves as a starting point in the scale of floods, and its adoption places the flood records of widely differing streams on a comparative basis. Special

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* *Proceedings*, Am. Soc. C. E., April, 1925, Papers and Discussions, p. 680.

Form No. 1.   Am. Soc. C. E. Research Committee on Flood Protection Data.   **FLOOD DATA**   Sheet No..........of........Sheets.

STREAM..
Location..
Drainage Area...........................Sq. Miles.
Source of Data...

Grading of Data........................   Total Length of Period of Daily Records ....Years.
(for period of daily records)   From.................to..............and from...............to............

GREATEST FLOODS

Greatest known flood in..................years..................c.f.s. = ...c.f.s. per sq. mile on....................
Second largest known flood..................c.f.s. =..................c.f.s. per sq. mile on....................
Third largest known flood..................c.f.s. =..................c.f.s. per sq. mile on....................
The greatest flood is known to be the maximum in at least....................years.

TABLE 1.   **FLOODS DURING PERIOD OF DAILY RECORDS**
Including all flows equalling or exceeding the "one-year flood" arranged chronologically

Ref. No.	Date.	Gage Height (feet)	Flood Disch. c.f.s. P=Peak rate. A=Aver. rate (24 hrs.).	Ref. No.	Date	Gage Height (feet)	Flood Disch. c.f.s. P=Peak rate. A=Aver. rate (24 hrs.).
(1)	(2)	(3)	(4)	(1)	(2)	(3)	(4)
1				41			
2				42			
3				43			

Form No. 3.   Am. Soc. C. E. Research Committee on Flood Protection Data.   **FLOOD DATA**   Sheet No..........of........Sheets.

STREAM..
Location..
Drainage Area...................Sq. Miles

TABLE 2.   **FLOODS NOT INCLUDED IN PERIOD OF DAILY RECORDS**

Ref. Letter	Date	Gage Height (feet)	Maximum Discharge c.f.s.	Maximum Discharge c.f.s per sq mile	Grading	Source of Information and Remarks
(1)	(2)	(3)	(4)	(5)	(6)	(7)
a						
b						
c						

TABLE 3.   **CHARACTERISTICS OF CERTAIN HIGH FLOODS**
(in order of magnitude as far as practicable)

Ref. Mark *	Date	Gage Height (feet)	Maximum Discharge c.f.s.	Maximum Discharge c.f.s per sq. mile	Aver. Rate of Discharge (c.f.s.) for Maximum Day	Duration of Flood (days)	Total Discharge During Flood Acre-Feet	Total Discharge During Flood Watershed Inches
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)

* Same as in Table 1 or Table 2,--i. e., use the same number or letter for a particular flood as in preceding tabulations.

Form No. 4.   Am. Soc. C. E. Research Committee on Flood Protection Data.   **FLOOD DATA**   Sheet No..........of......Sheets.

STREAM...
Location..
Drainage Area........................Sq. Miles.

TABLE 4.   **DATA FOR PLATTING FLOOD HYDROGRAPHS**

Date.	Hour.	Gage Height.	Discharge	Date.	Hour.	Gage Height.	Discharge.
(1)	(2)	(3)	(4)	(1)	(2)	(3)	(4)

FIG. 16.—SAMPLE FORMS FOR COMPILING FLOOD DATA.

pains are taken to indicate by means of a symbol whether the figures refer to peak flow or average 24-hour conditions (Form No. 1, Table 1, Column 1) (Fig. 16). For convenience of reference the three greatest known floods are entered at the top of Form No. 1 (Fig. 16), irrespective of whether or not they occurred within the period covered by daily observations. The data so listed are fundamental for many studies. In order that its degree of reliability may be quickly judged, each record is graded with a number, the scale of which runs from 5 for data of the best character to 0 for information of questionable accuracy.

Form No. 3, Table 2 (Fig. 16) is designed for listing floods not included in the periods covered by daily records, but concerning which definite facts are known. It is not amiss to state here that early flood data of considerable value have been obtained from a variety of sources, and have proved of much value in determining the frequencies of recurrence of great floods. On Form No. 3, Table 3 (Fig. 16) may be tabulated in greater detail several of the floods listed in Table 2.

Finally, on Form No. 4, Table 4 (Fig. 16), may be entered data for platting flood hydrographs, consisting of both gauge heights and discharges. This form was adopted by the Committee in preference to graphic representations, because no one form of hydrograph could be devised that would answer the many different requirements of engineers, and it was deemed best to leave the plotting to individual tastes. All the forms were designed to be capable of indefinite addition and expansion as future data are obtained.

These forms and the data to be entered on them represent the first step in the program of the Committee, the next step being the condensation of them into diagrammatic or other form for general use and reference. To date, 139 streams have been compiled on the forms, thanks to the co-operation received from various sources. Engineers possessing flood data will confer a favor on the Society and on the profession as a whole if they will co-operate with the Special Committee on Flood-Protection Data in this important work.

FORD KURTZ,* M. AM. SOC. C. E.—Flood problems can be divided into two major groups, namely, those requiring for their solution knowledge of flood heights under existing physical conditions and those requiring the estimation or prediction of flood heights under other than existing conditions. The solution of all problems coming under the latter classification depends on the rate as well as the height of flood discharge.

Nearly all flood investigations required in highway and railroad design belong to the first group, since, in general, only slight channel encroachments are involved. Information as to the maximum stage of streams is often obtainable when data on the rate of discharge are completely lacking, so that highway and railroad engineers can usually secure the records they need, whereas the designer confronted with problems in the second group finds in many cases only very meager data to aid in their solution. Thus it happens that irrigation, public water supply, and hydro-electric engineers, who nearly always deal with design problems of the second group, are much more deeply

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* Care, J. G. White Eng. Corporation, New York, N. Y.

involved in the study of stream flow characteristics than either the highway or the railroad engineer. However, the final difference in the difficulties encountered by the two groups of engineers in their flood-control problems is not quite so great as indicated, by reason of the fact that highways and railroads involve many streams in a single project and often pass through regions where irrigation, public water supply, and hydro-electric developments could not be financed, since bankers are loath to sponsor projects of which water supply is the basic factor unless stream flow records are available for a period of years, or the discharge can safely be estimated from records on adjacent streams. Thus, some observations of flood discharge as well as of flood heights are usually available for solving problems of the second group.

This paper presents in both tabular and graphic form a great amount of data on maximum recorded flood discharges and in a general way interprets them and indicates their application to specific problems. The latter feature is the chief measure of the value of the paper, since it determines the practical use. Plate VIII,* which summarizes the data graphically, shows such great variations in the maximum recorded flood discharges for a given area of drainage basin as to create at once the impression that the author has consolidated data too diverse (both geographically and in the period of observation) to permit of drawing conclusions of value. Furthermore, the method adopted in preparing Plate VIII takes no account of the frequency of floods of different magnitudes, which is often an important factor in design.

Geographical diversity, as representative of different hydrological conditions, can be taken care of by preparing more diagrams from a given number of observations, but the correction for difference in length of period of observation, which is at least equally important, is not so simple. For this latter purpose, the speaker proposes the analysis of flood flows by the probability method. He considers the practical application of the mathematical theory of probabilities to the problem of frequency and magnitude of floods the most noteworthy single contribution of the last twenty-five years to the science of hydrology. Its uses are many, and it is the best method for correlating flood records having different lengths and made during different periods of years, as well as for determining probable frequencies of various sizes of floods, where such data are needed for purposes of design.

If the records presented by the author were first segregated into at least as many geographical divisions as are listed on Plate VIII and if certain representative streams of each division, beginning with those having the longest records, were then studied intensively by the probability method, the plotting of the resulting values for the probable 100-year or 1 000-year flood would undoubtedly show a tremendous decrease in the width of the "belt" of points as compared with Plate VIII. This method of studying an individual stream often results in the discovery and rejection of erroneous or doubtful data, which might otherwise go unnoticed, and will always smooth out in a consistent manner the irregularities of records on the stream. Thus, it may happen that the maximum flood observed in a 10-year period may actually be one which occurs only once in every 100 years on the average. It

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* *Proceedings,* Am. Soc. C. E., December, 1924, Papers and Discussions, p. 1555.

is truly remarkable how the probability method will show such a flood to be "out of line". By that it is not meant that the flood could not have occurred, but rather that it will appear quite plainly as a 100-year flood which chanced to occur in the 10-year period of observation. No other method will bring out these features so clearly. The speaker, therefore, believes that intensive and careful study of a fewer number of streams in the manner just described would be much more valuable than the compilation of a mass of data on maximum recorded flood flows so great as to preclude detailed study of them.

There still remains for evaluation the important factor of shape of drainage basin, which causes wide variation in the rate of flood flow from basins having the same area and subject to the same hydrological conditions. This is a matter of individual examination for each stream. Obviously, the effect of this factor can be studied much better if geographical diversity and difference in period of observation have already been eliminated in the manner suggested.

The speaker by no means wishes to disparage the use of general curves based on the records of a number of streams, but he does feel that such curves must be used with great care. In general, if as many as ten years of flow records for any stream are available, intensive study of the higher recorded flows of the stream itself is often more valuable in predicting flood frequencies than curves based on longer records of adjacent streams. Some engineers might feel that a 10-year period of observations is too brief to serve as a basis of probability studies. However, examination of the hydrograph of any stream will show a great many peaks projecting above the general level, rather than one only. In the speaker's opinion these should all be classified and studied as floods, the problem being to fix the "basic stage" for a flood, that is, the line between those flows governed by a few well-defined factors and those governed by the laws of probability and chance. If this opinion is correct, ten years of records may yield, not ten, but twenty, thirty, or even forty, values for probability determination. These values will give a very good idea of the trend of floods—not the true trend, to be sure, but an indication of it that represents a great advance over the results of any other method thus far presented for the study of flood problems.

In order to fix this dividing line, or "basic stage", for floods on any given stream, it is only necessary to select various values, assume every peak above that value to be a flood, and from the points thus obtained derive a probability curve. That value of basic stage the points of which lie nearest to a straight line on probability paper is chosen as the correct value, and the resultant straight line as the flood probability curve.

As already mentioned, many design problems require a solution based on the frequency of floods of different magnitudes, because it is not always necessary or even desirable to design structures in such a manner that there will be no damage caused by a large, infrequent flood. One of the great advantages of the probability method is that, more than any other, it presents a quite definite idea of frequencies as a guide in the two major features of flood-control design, that is, the economic and the social. The economic feature is embodied in the question, "Will it pay from the standpoint of probable

property loss over a long period of years to protect against certain floods?" Here the frequency studies are of utmost value. The social feature involves the safeguarding of human life and the frequency studies point the way to the determination of the flood having such a small percentage of probability of occurrence that there is practical assurance it will never happen.

The speaker, therefore, wishes to urge the general adoption of the probability method as the best presented thus far for the study of compiled flood data, such as those submitted by the author, and for the study of the individual flood problems confronting all hydraulic engineers.

CHARLES W. COMSTOCK,* M. AM. SOC. C. E.—The author has rendered a valuable service in collecting and arranging the records of about 1 000 flood discharges from drainage areas of all sizes in different parts of the world.

The speaker believes, however, that it is a mistake to relate the rate of run-off to the area of the drainage basin. This is, to use a crude analogy, as if one sought an empirical formula for stress in a beam in terms of the length as the only argument, leaving the load, the cross-sectional dimensions, and the method of support to be covered by an all-inclusive coefficient to be guessed at subsequently.

The author enumerates a considerable list of variables, both meteorological and topographic, which influence flood discharges, and then selects one (probably not the most important), "as the logical basis for a general formula", ignoring the others except as they may be included in a coefficient the values of which range from 100 to 10 000. Such procedure is certainly not scientific, nor is it likely to throw any light into the dark corners of this complicated subject.

It may well be doubted whether a formula is worth while in view of the large number of variables, most of which it is impracticable, if not impossible, to evaluate for any given drainage area. Careful study of Table 2† will give quite as much information and a far better understanding than can be had from any formula that attempts to summarize the data. Moreover, the very existence of a formula is likely to lull engineers into a comfortable, but unjustified, belief in the precision of results, instead of stimulating that healthy skepticism and critical examination of every step which so often keep them from serious blunders.

As already noted, the speaker believes that the area of the drainage basin is probably not the most important factor in determining flood run-off. The author states that he considers it the "logical" variable to which run-off may be related. May he not have been drawn to it, perhaps subconsciously, because it is the only factor which can ordinarily be ascertained?

The area covered by a violent storm is relatively small. A limited drainage basin may get the full effect of such a storm over its entire area at one time. A large basin may be crossed by the storm over a short line or a long line. In the latter case the total precipitation on the basin may be large, but it does not fall on all parts simultaneously. If the path of the storm is down

stream the cumulative effect may build up an enormous flood from a moderate rate of precipitation. If the path is up stream a violent storm may produce only ordinary high water. Again, a persistent and prolonged rain covering the entire basin at one time may cause a greater flood than either of the storms mentioned, although the total volume of water falling may be no greater. In these three assumed cases, all of which are not only possible but of frequent occurrence, the area and all other features of the water-shed are identical, and the total precipitation in a given time may be the same, yet the discharges at the gauging station will differ greatly.

An element of great importance (perhaps more than the area itself) is the permeability of the underlying rocks. The author mentions this only casually. This point was first emphasized by Belgrand in his studies on the basin of the Seine. He pointed out that 75% of this basin is underlain by permeable rocks, the annual discharge of the river being only 28% of the precipitation; for the Garonne, of which about one-half the basin is impermeable, the run-off is 52%; and for the Po, the basin of which is almost wholly impermeable, the run-off is 64 per cent. These basins are of nearly equal size, that of the Seine being 30 000 sq. miles; that of the Garonne, 33 000; and that of the Po, 27 000.

A peculiar feature of the Po floods is that the discharge is nearly the same at all points along the lower 150 miles of the river. This seems to be due in part to the lack of synchronism of flood discharges on the numerous tributaries, and in part to the enormous storage capacity of the endiked channel.

The upper part of the Arkansas River in Colorado lies between the Continental Divide on the west and the Mosquito Range to the east. At the time of the spring run-off and consequent flood stage of the Arkansas, almost the entire supply comes from the tributaries which drain the granite massif of the main range, while the limestones, sandstones, and other permeable rocks of the Mosquito Range contribute only a trifle to the gulches which feed the river from the east.

It seems impossible that the various elements here mentioned, as well as many others, should receive suitable quantitative designation and be incorporated in any formula for general use. The engineer will necessarily be driven back on his judgment of the significance and importance of each factor in a given case, and he might better apply that judgment directly to the tabulated data.

It is easy to be critical. As Byron said:

> "One must serve his time to every trade
> "Save censure. Critics all are ready made."

It is another and far more difficult thing to contribute a new fact or a new principle. It is important, however, that in the desire to render knowledge more useful by classifying and formulating it engineers should not jump to hasty generalizations which may mislead.

As the author points out, the gauge height at flood stage is more important than the volume of discharge. It matters little whether the quantity of water flowing is large or small, as long as the surface does not rise high enough to

endanger lives or destroy property. It is well known that on the front of a flood wave the maximum velocity, maximum discharge, and maximum gauge height occur in succession in the order named. Thus, two flood waves of equal maximum discharge may have quite different gauge heights, depending on the steepness of the wave front.

There is a tendency to think that floods become increasingly destructive as time goes on. This is not so much because floods are greater in volume now than formerly as it is that there is more to destroy. For instance, the Seine flood of 1910 had a maximum gauge 0.39 m. less than that of 1658, but did incomparably more damage for an obvious reason.

The author's warning against the misuse of calculated flood frequencies is worthy of careful attention, especially in view of the prevalent, and in the speaker's opinion improper, tendency to base probability calculations on small numbers of observations.

In 1873, Belgrand published his monograph on "The Seine", one of the most elaborate and exhaustive treatises ever devoted to the study of a single river. In discussing the flood of 1658, he wrote:

"The meteorological phenomena which produced the flood of 1658 have manifested themselves again in our day. They have been renewed three times between 1732 and 1872 (140 years) in the floods of 1784, 1836, and 1866, or once every 47 years, and they were separated by intervals averaging 35 years. But, in order that there should result a flood similar to that of 1658, it is necessary that they should occur not more than five or six days apart. The probability of such an approximation is very small, and, by calculations which I think it useless to reproduce here, I have demonstrated that the recurrence of the flood of 1658, under the same conditions, i. e., by floods in only two of the affluents, would require thousands of years."

Thirty-seven years later, in 1910, it happened.

A probability, however logically derived, is not a forecast. It is certain that what has occurred may recur, and even worse conditions may present themselves. There comes a point beyond which it is not financially practicable to go in providing protection against possible but improbable floods. At this point the engineer, like all good gamblers, must be willing to risk his money on a "long shot".

THADDEUS MERRIMAN,* M. AM. SOC. C. E.—The speaker has been interested in floods for many years. He was fortunate in getting on to the ground in the Ohio Valley very shortly after the flood of 1913 and has kept up a lively interest in all matters pertaining to floods ever since. This problem is one that will always be with us because every time a new flood occurs it becomes necessary to revise a large part of our so-called fundamental data.

The speaker is heartily in accord with the point of view presented by Mr. Comstock.† This matter of floods is not one which can be fairly treated by any method of probabilities. Maximum flood discharges are related to the duration of the storm and its intensity, to the direction of its path, to the characteristics of the topography and of the geology of the water-shed, far more than to any simple function of the area of the drainage basin.

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On account of the ease with which these problems can be discussed with pencil and paper there is a tendency to neglect one source of information respecting flood heights that is worth more than all the direct observations which have been accumulated to date.   This information is to be found in the records which the floods themselves have left on the walls of the valleys they traversed.   This does not refer to the drifts of wood and leaves or other similar perishable substances, but to the sand and gravel bars which lie above the stream bed.   They are to be found in every river valley and are better indices of past flood heights than any so-called records as to observed maxima. No analysis of possible flood heights can be considered complete, which does not include the records which the great floods of the past have engraved on the topography.

If, on the Catskill drainage area, there had been used any method of 10-year periods and had a probability curve been developed from them, the results would have been shattered beyond recognition by the flood of October, 1924; and that flood, moreover, was far from being a maximum.

There is a fundamental reason for the anomalous position in which we find ourselves with respect to this problem of floods.   There are existent very few dependable records prior to 1900.   Many floods occurred before that year and many have since occurred and passed without being recorded.   Most of what are called flood records are not worthy of the name.   Table 2* as presented is not a record of floods; it is merely a tabulation of observed stream flows.   This is a fact which must be held clearly in mind, else we shall continue the vain task of studying and analyzing a mass of data that bears no known relation to real floods as such.

WESTON E. FULLER,† M. AM. SOC. C. E. (by letter).‡—The paper calls attention again to the importance of the flood problem and sounds another warning of the inherent danger which lies in the encroachment of the channels that streams have used in the past for the purpose of carrying the great floods which come only at rare intervals.   The great disasters that have come to many communities in the United States show this danger clearly.   That similar disasters will occur at many other places can scarcely be doubted.   The mere fact that for many years streams have not experienced floods that have caused any damage is, in itself, no assurance as to the future.   The unusual conditions that produce great floods may occur on any stream, just as, in the past, they have come on other streams, and floods greatly in excess of the usual ones are possible at any time.

The author has discussed a number of characteristics of floods in a very helpful manner.   He proposes a formula which, he concludes, represents the effect of the size of the drainage area on the maximum flood.   Many investigations have been made for the purpose of determining this effect.   As a result of these studies, a number of formulas have expressed the relation as an exponential function, in most of which the relation was found to be quite different from that now proposed by the author.

Grouping all the other factors affecting floods into a coefficient designated C in each case, the formulas resulting from these investigations may be expressed, as follows:

$$Talbot \quad \ldots\ldots\ldots Q = C\,M^{0.75}$$
$$Fanning \quad \ldots\ldots\ldots Q = C\,M^{0.83}$$
$$Burkli\text{-}Ziegler \quad \ldots Q = C\,M^{0.75}$$
$$Dickens \quad \ldots\ldots\ldots Q = C\,M^{0.75}$$
$$Cooley \quad \ldots\ldots\ldots Q = C\,M^{0.67}$$
$$Ryves \quad \ldots\ldots\ldots\ldots Q = C\,M^{0.67}$$
$$Dredge \quad \ldots\ldots\ldots Q = C\,M^{0.67}$$
$$Metcalf\ and\ Eddy \ldots Q = C\,M^{0.73}$$
$$Fuller \quad \ldots\ldots\ldots Q = C\,M^{0.67} \text{ for small streams, increasing to}$$
$$Q = C\,M^{0.8} \text{ for very large rivers.}$$
$$Cramer \quad \ldots\ldots\ldots Q = C\,M^{0.7} \text{ to}$$
$$Q = C\,M^{0.8}$$

The author's proposed formula is $Q = C\,M^{0.5}$, which is quite different from the previous formulas and which calls for an analysis of his data and methods in order to ascertain his reasons. He derives his formula by plotting the maximum rate of flow of many different streams, and draws a curve to envelop the highest of these maximum flows. The writer believes that this method of treating flood data has a fundamental error which leads to an erroneous conclusion. This opinion the writer has previously expressed in discussing other formulas derived in a more or less similar manner. In the closing discussion of his paper on "Flood Flows,[*] the writer made the following statement:

"* * * other formulas, derived by plotting the maximum floods which have occurred on streams of different sizes, give the greatest flood which has occurred on any of the large number of streams in the varied intervals covered by the several records.

"As there are many more small streams than large ones, it is obvious that there are more chances of obtaining an extraordinary flood on some one of the many small streams than there are of obtaining a similar flood on one of the few large ones."

This criticism was offered respecting data selected for streams which had similar characteristics so far as slope, rainfall conditions, and other elements affecting floods are concerned, and was simply directed toward the greater probability of securing relatively great floods on the large number of small streams than of securing a similar result from a few large rivers. In his paper, however, the author has not confined himself to a consideration of streams of a similar nature as to other characteristics affecting floods, but has plotted on his diagram (Plate VIII),[†] the extraordinary floods which have occurred on a few of the many small streams that are, in the main at least, mountain streams, and has compared these floods with those on the few large rivers on which no similar conditions exist.

In this case, therefore, the data are unbalanced in two different respects, and it is no wonder that the effect of the size of the catchment area is greatly exaggerated. An examination of Plate VIII will show that the enveloping curve is controlled at the upper end by a few points, all of which are extraor-

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 681.

† *Proceedings*, Am. Soc. C. E., December, 1924, Papers and Discussions, p. 1555.

dinary floods which have occurred on a few streams having unusually favorable conditions for great floods so far as their drainage areas are concerned, and that these floods represent most extraordinary occurrences among thousands of similar streams. The other end of the curve is controlled by the floods which have occurred on the few large rivers in the United States. It will be noted that these rivers do not have similar characteristics so far as slope and other conditions are concerned. The author's formula, therefore, covers the extreme conditions for small streams, but only ordinary conditions for the large rivers, and does not properly indicate the effect of size of drainage area.

This is well illustrated by considering the two extreme points on Plate VIII. Beacon Creek, the smallest stream plotted, has a catchment area on the side of a steep mountain, most of which is impervious; from it the run-off is very large. At the other end of the diagram is the Mississippi River, on which the conditions are nearly opposite. That the difference in run-off per square mile between these two extreme cases is largely effected by the character rather than the size of the water-shed, cannot be doubted.

Bearing in mind that the author has plotted on Plate VIII floods which have occurred on nearly all the large rivers in the United States, it would be well to consider what the diagram would look like if he had followed the same course with small streams. In such a case he would have had a mass of points plotted on the left side of the diagram, representing the largest floods on the enormous number of small streams that have never had a flood as great as those plotted. A curve through the center of these points representing average conditions on small streams, and through the center of the points for the larger rivers, would indicate the effect of the drainage area.

Methods of treating data similar to that used by the author are quite common. For this reason the writer believes that it is well worth while to discuss the matter in considerable detail in order to show the reason why erroneous conclusions may be drawn. It may clarify the subject somewhat to indicate what would have resulted if a similar method had been followed in deriving other well-known formulas.

For instance, consider the formula to which the author likens his own, that is, the Chezy formula for the flow of water in conduits. Suppose that, in deriving the Chezy formula, data had been used which represented the maximum flows obtained in any conduits, irrespective of their slope or surface conditions, and that the formula had been based on the maximum points obtained by plotting these results. The result would have been a curve controlled at one end by the flow in small conduits with smooth surfaces, and, at the other, by the flow in large rivers, for which much less slope would have been found, and for which the surfaces would have been rough. It is needless to say that the formula thus obtained would have given the relation between the velocity and the hydraulic radius very different from that which was obtained by comparing conduits of similar slopes and similar surface conditions. Such a formula would have been misleading indeed if applied to similar conduits of different sizes. The author has used a similar method in obtaining his formula. It is essential, in using data of this kind for the

purpose of ascertaining the effect of any element, that the effect of all other elements be made at least similar, otherwise the results do not show the true relation.

The author tabulates the maximum floods for a large number of streams, the information having been collected from many different sources. These data are useful, and Mr. Jarvis has performed a real service in bringing them into a single table (Table 2).* Any one who has traced data of this type to their sources, must have been impressed with the likelihood of errors, as the items are transferred from table to table. Flood data are particularly unreliable in this respect.

In the first place, the records of these floods are of varying degrees of accuracy. Some of them are undoubtedly close approximations to the actual peak flows of the floods in question; others are probably far from the actual run-off. This must be true for many of the floods, particularly for those which have caused great destruction. In the great majority of such cases the discharge has been estimated by the use of the Kutter formula. The data for determining this flow have been obtained subsequently by observation of the water marks of the maximum height attained by the flood at any time during the flood period. Whether or not the slope thus obtained represents the slope of the water flowing in the channel cannot be ascertained with any degree of accuracy where the channel has been materially changed by the floods. Moreover, the value of the coefficient, n, in the Kutter formula at the time the maximum height was attained at the point under consideration, is doubtful. It is impossible to say how much the flow in the channel is being retarded at that particular time by the mass of débris flowing down the stream. It is impossible to know whether or not the height which the flood reached at that point was caused by water flowing in a free channel or whether it was backed up by submerged obstructions. Little is known of the effect on the flow of great masses of soil, trees, brush, rocks, and other material which are being carried down the stream by these floods.

Mr. Winsor, in his discussion,† vividly sets forth the condition that exists in some extreme cases where the water is practically pushing the large mass of matter ahead of it. Is it not true that for most destructive floods similar conditions, to a lesser degree, occurred at some time during the flood, thus establishing high-water marks which are afterward taken as the slope of the water which is assumed to have been flowing freely in the channel as left after the flood recedes? It must not be overlooked that, in estimating the discharge for many of the floods, values of n in the Kutter formula as low as 0.030 have been used, and seldom, so far as the writer can ascertain, have values of more than 0.040 been taken.

The writer is inclined to think that for most destructive floods the discharge has been over-estimated. In tables such as Mr. Jarvis has presented, as well as in many similar tables, there is no indication of the accuracy of the data, yet from the original source of the information, it is found—in many

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* *Proceedings,* Am. Soc. C. E., December, 1924, Papers and Discussions, pp. 1563–1579.
† *Loc. cit.,* May, 1925, Papers and Discussions, p. 901.

cases at least—that the engineer who estimated the flood had a clear conception that his estimate may have been far from the actual flow.

As an instance of this, consider the floods on Cane Creek, in North Carolina, and on Elkhorn Creek, in West Virginia, two of the largest, relative to the drainage area, of which there are records. The quantity of water flowing during the peak of these floods was estimated by Mr. E. W. Myers, who described* the difficulties of ascertaining the discharge during these floods, and frankly stated that his estimates might have been far from the truth. It appears from his description that in estimating the flow he used the slope of the channel as it existed after the flood had receded.

To illustrate the nature of these data it may be well to quote a little of what Mr. Myers has said in regard to his measurements. As to Cane Creek, he says:

"For these reasons the writer has used all the means in his power to obtain such information and it is presented below, though with diffidence, for the possibility of considerable error is great, but the results are the best that can be obtained under the existing circumstances.

"* * * . Unfortunately, no gaugings of this stream at anything approaching flood heights has ever been made, so that a rating table giving the relation between height and discharge cannot be constructed in the usual manner. Failing in this, therefore, there are two approximate methods which can be used for obtaining this information, one based on the consideration of the amount of the rainfall during the period, its distribution, the condition of the earth as to saturation, etc., together with the shape of the basin, its average slope and the general topography; the second based on the cross-section of the channel at some point, its condition as to roughness and the slope for some distance above and below the point where the cross-section is taken.

"* * * . This method is open to objection on the ground that there is great difficulty in selecting the proper value of n, and also from the fact that the slope at flood stages is probably not the same as at low water, except at the crest of the flood, when the water is neither rising nor falling."

As to Elkhorn Creek, he says:

"As stated before, in this way no estimate of the maximum flood volume is possible and we are compelled to resort to estimates based on the slope and cross-section of the channel. Such estimates in this case present even greater difficulties than in the case of Cane Creek, since the waters of Elkhorn Creek were much more obstructed and the cross-section at different points is much more variable, making the coefficient of roughness greater than before, and introducing more uncertainty as to its proper value. As the method used was precisely the same as that used on Cane Creek, the calculations will not be given here in detail as on that stream."

Other floods, which rank high among the extraordinary recorded floods, are those which occurred in the vicinity of Philadelphia, Pa., in 1843. The record for the discharge during these floods is found by a recent re-computation based on data obtained at the time of the flood. The writer is not criticizing either the accuracy of the data taken at that time, or the method of computation, but simply noting that there is a large chance of error, and that too much dependence cannot be placed on such data. For many of these extraordinary floods, therefore, there is merely an estimate based on doubtful

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* *Engineering News*, Vol. XLVIII, August 7, 1902, p. 102.

data, with no assurance that these estimates in different cases are properly comparable.

It appears further, in comparing the various tables of maximum flood flows, that there are considerable differences as to the magnitude of these floods in the different tables. In some cases these differences are accounted for by revisions of the original estimate; in others they probably are due to typographical errors and similar causes. Each time a new table is presented, some of these errors creep in. Evidently, such data are in much confusion, and there is real need of a careful study of the original records of all these great floods in order that their relative reliabilities may be determined.

The author's proposed method of determining the maximum flood on a stream is evidently based on consideration of the single flood on that river. For many of the rivers there is no method of telling whether or not this single flood is an extraordinary event. It is true that, for many streams, the author has listed what he indicates as frequent and rare peaks. So far as the writer can ascertain, these frequent and rare peaks are taken from two or three different sources. Most of them represent the data taken from the writer's paper on "Flood Flows"; others are taken from the study of floods in California, and a few others from observations and estimates by the author. So far as those taken from the writer's paper are concerned, the frequent peaks represent the yearly floods as obtained by averaging the largest flood for each year, over a period which was then available. The end of this period was about twelve years ago, and more reliable data must now be available for many of these streams. The rare peak merely represents the largest 24-hour average flow during that period.

The basing of any estimate on the single maximum flood, without consideration of any other floods which have occurred on the stream, leaves the time element entirely out of consideration.

Mr. Lane's excellent statement* of the meaning of flood frequency and its importance, will serve to clarify this subject. His study, which is confined to New England streams, indicates a frequency relation for flood flow expressed by $Q = Q_1 (1 + 0.69 \log T)$. The writer, basing his study on all the streams of the Eastern Coast, which included, of course, Southern as well as Northern rivers, found a relation as to frequency for those rivers in accordance with the formula, Q maximum $= Q$ average $(1 + 0.75 \log T)$; for other parts of the United States, he found a larger variation in respect to T and adopted as a general average, $1 + 0.8 \log T$, believing that the data then available did not justify any attempt to use a different relation for different localities.

In making his study, Mr. Lane has used a method which gives the flood to be equalled or exceeded, corresponding closely to that described by the writer.† Using Mr. Lane's illustration of a series of 1 000 floods on a stream, he is correct of course when he states that the one-hundredth of the series is that which will be equalled or exceeded on an average of once in a 10-year period. But what about the largest flood? It is not the flood which will be equalled or exceeded in 1 000 years. It is the flood which will probably

occur. It is the flood for which there is an equal chance that it will occur in another thousand years. This is what the writer's formula gives, and, in his opinion, it is what is desired. With Mr. Lane's method, this largest flood cannot properly be included in his series unless he includes it for a period of 2 000 years. From the method outlined in his discussion, it would appear that he has not done this, but has taken the largest flood as the one 'which is equalled or exceeded for the full period. If so, he is slightly in error for about one-half his records.

If it were desired to find the flood for which to design, so as to be quite sure that the structure would fail within a given period, then the flood that is equalled or exceeded in the period would be the proper one to use. The writer's view of the matter, however, is that for the design of the structure, it is desired to ascertain the flood for which there is an even chance that it will not fail during the period, and for that reason he used the average flood. It really matters little which method is used, if the computer understands what he is doing. Substantially the same results would be obtained if the value of T in the Lane method is made twice as great as in the writer's method. The relation between these methods has been discussed previously[*] at some length by the writer.

Mr. Lane apparently obtains his yearly flood by plotting the data rather than by averaging the floods. There is much to be said in favor of this method and something to be said against it. As he states, it eliminates any consideration of the larger floods and, in cases where these floods are unusual, this result would be better. It is not certain, however, that these larger floods are unusual. Is it not equally possible that, for the period in question, the stream has had an unusual number of smaller floods? The writer considered this question at the time he prepared his paper. He was much inclined to adopt a method similar to that proposed by Mr. Lane, but concluded that, while in some cases the average was unduly affected by including the larger floods, the error in neglecting them, while less apparent, was, in other cases, quite as great. In his own work, however, he takes into consideration both the actual average yearly flood and the yearly flood as indicated by plotting. For long records either method will produce satisfactory results, while for short records there is, of course, bound to be some error.

Mr. Lane has computed the probable error in obtaining the one-year flood by means of records of different lengths, based on the Merrimac record. His results seem to agree quite closely with those found by the writer from a study of many different rivers to ascertain the probable error in finding the average yearly flood.[†] In so far as the probable error is concerned, there seems little to indicate which method is the better.

A study of the floods on streams where large ground storage exists, or where there are considerable storage reservoirs which are partly emptied at times when floods occur, has convinced the writer that in some cases a method almost the opposite of that suggested by Mr. Lane is necessary. For such streams the floods are divided into two series, one representing the floods that come

[*] *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 564.
[†] *Loc. cit.*, Fig. 19, p. 685.

when storage is to be had in the ground or in the reservoir, and the other including the floods when no storage is available. The plotting then may become two parallel lines, one distinctly higher than the other. This is because the storage, when it is available, affects the total run-off from large and small storms by about the same amount, thus being a much greater proportion of the small storms than it is of the larger ones. In such conditions the flood to be used in the place of the yearly flood in the formula, should be determined largely by consideration of those floods which occur when little or no storage is available. It can be approximated by considering only the upper points representing the larger floods, drawing through these points a line representing the frequency relation and considering the "yearly flood" as that given by the intersection of this line with the one-year line. This, of course, may be higher than the average and much higher than that obtained by the Lane method. While such cases are not common in most sections of the United States, yet in a study of Southern California streams several were found, and the influence is undoubtedly felt for other rivers, although it is less marked. Here the method proposed by Mr. Lane would lead to a much larger error than the use of the average flood.

In arid regions these conditions are exaggerated, so that no floods of any moment occur during many years, yet large floods sometimes come. In his paper on "Flood Flows," the writer stated:*

"* * * in arid and semi-arid regions * * * conditions are such that no floods worthy of the name occur in some years, though occasionally there is a large one. For such cases the average yearly flood becomes low and the percentage of the maximum flood becomes unduly high. These conditions are quite different from those generally found, and it seems best to exclude such cases. It follows, of course, that the frequency relation here proposed does not hold for such rivers."

Mr. Lane suggests that the relation between the average 24-hour flood and the maximum rate of flood, as included in the writer's formula, is the weakest part of the formula. The writer agrees. At the time the study was made little data were available on the maximum rate of flow, only occasional records could be found for most streams, and there were few continuous records. What data were to be had consisted, in the main, of records based on the 24-hour average, so that it was necessary to make the study on that basis. The writer appreciated the fact that the peak flow for small streams would be a larger proportion of the 24-hour average flow than for large streams, and established the relation as best he could with the data at hand.

At present, there is a large amount of additional data available, and a far better relation can undoubtedly be established. This the writer hopes to do at an early date. Mr. Lane suggests that, instead of using this relation, his Q_1 may be used. With this conclusion the writer is agreed, providing that it is possible to obtain directly the flow at the peaks of the floods. At the time the writer proposed his formula, it was quite impossible to obtain these peak flows for any considerable number of rivers. Even to-day the bulk of the data is still expressed in terms of a 24-hour average, so that the relation between

the peak and the 24-hour average is a necessary part of the formula. Where, however, the peak flows are available, the writer certainly would agree that they should be used in preference to any formula which could possibly be devised. In other words, where facts are available, they should always be used in preference to any indirect figures obtained by means of formulas.

There is one point in regard to this relation between maximum rate and 24-hour average rate to be made clear. Mr. Lane criticizes the relation because it does not agree with some individual cases. The relation as expressed is intended to be that between the maximum flow at the peak, during a period of years, and the maximum 24-hour average, during that same period. This relation cannot be expected to hold for individual floods, as the two maximums do not necessarily come during the same flood.

A few years ago the writer made a partial check on the frequency relation in his formula by again plotting the data for selected streams with long records, including therein the floods for five or six additional years. For most of these streams the indications were that the individual plottings approached more closely to the general relation than previously. With the large amount of additional data now available, it may be that frequency relations for different sections of the country are justified. It is the writer's intention to make a re-study of this phase of the problem.

The investigations which have been made have only scratched the surface of the flood problem. The maximum peak flow, although an important element in many problems, is in others less important than the maximum total flow during flood periods. In fact, the hourly flow for small streams, representing as it does the bulk of the run-off, is perhaps properly comparable to daily flows of larger streams, weekly flows of still greater rivers, and flows of several weeks for the Mississippi River in its lower stretches. Each represents the run-off for somewhat similar conditions. A study along such lines might tend to clarify some points not yet understood. There have been many studies of the effect of such elements as the average slope of the drainage area, the relation of the length to the width of drainage area, the average rainfall over the area, and some other elements. The effect of the percentage of the water area of lakes and ponds is another factor which could well be studied. Information as to the effect of these elements, as well as to the effect of the size of the drainage area and of the frequency relation, would place in the hands of the engineer tools with which to work in solving the problem for individual streams; but they can never eliminate the necessity for the careful study of the stream itself or the exercise of judgment.

Formulas which confuse the effect of any element are, however, not useful tools, as they tend to confuse the judgment. In the writer's opinion, the author's formula, purporting to show the effect of drainage area, while in fact including thereunder the effect of many other elements, comes in this class.

H. DE B. PARSONS,* M. AM. Soc. C. E. (by letter).†—This paper calls attention to a most important subject, and the data of maximum observed flood

discharge rates given in tabular form will be useful to those studying flood problems.

The maximum run-off in any district must be strongly influenced by the characteristics of the district, such as the geological formation, surface contours, number and size of tributaries of the main stream, forestation, etc. When the area of a drainage basin is large, the physical characteristics of part of the area may (and often do) differ widely from those of other parts.

If the flood discharges given in Table 2* were segregated into groups according to physical characteristics and to geographical location, the variation in the value of constant, C, between upper and lower limits, as shown in Plate VIII,† would be greatly reduced for each group. This would make the practical application of the formula less liable to error.

It would seem that a formula to express the discharge in second-feet as a maximum to be expected, based on known flood records, should be of the form,

$$Q = C \, M^{\frac{n-1}{n}} \, K$$

This is the author's Equation (2),‡ with the addition of K, a variable to cover the physical and geographical characteristics of the drainage area. It might be possible to determine values for K by classifying the data in Table 2, and platting the points in a manner similar to that used for Plate VIII. Then, K would be constant for each classified group, and C would be constant for all groups, but sufficiently large to cover the maximum expected floods.

The writer suggests that this matter be referred to the Society's Special Committee on Flood-Protection Data and to the author, who deserves great credit for the work he has done. Perhaps a detailed study of classified drainage basins would develop the fact that values might be determined for K which would be characteristic for certain physical and geographical conditions and which would bear some simple relation to the discharge corresponding to known precipitation. This idea, expressed mathematically, might take the form of:

$$K = \text{Function of } \frac{\text{Discharge}}{\text{Corresponding precipitation}} = f\left(\frac{D}{P}\right).$$

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* *Proceedings*, Am. Soc. C. E., December, 1924, pp. 1563–1579.
† *Loc. cit.*, p. 1555.
‡ *Loc. cit.*, p. 1553.

ELEMENTS GOVERNING THE DEVELOPMENT OF HIGHWAY TRAFFIC

Discussion*

By Messrs. Arthur W. Dean and Van Alen Harris

Arthur W. Dean,† M. Am. Soc. C. E.—The prime origin of the extensive expansion of highways and the increase of highway traffic is the development of the internal combustion engine, just as the development of railroads and street railways in the Nineteenth Century originated with the improvement of the steam engine and the electric motor.

An outstanding factor of development in the United States, and one that is usually overlooked, is the general prosperity and aspiration of the people as a whole.

As in the past, various other elements will affect or govern the development of highways and the traffic over them, and some of these elements, although possibly mentioned by other writers, are so applicable that they will bear repetition:

1.—The gradual increase in efficiency and decrease in cost of motor vehicles leads directly to an increased use of such vehicles for the transportation of freight on what are commonly termed short hauls, and also toward an increase in the economical hauling distances.

2.—As the use and improvement of the motor vehicle progresses, the railway and railroad lines that are now being used mainly for short hauls may be outrivaled and their use discontinued, thereby increasing highway traffic.

3.—Many industries are now located at points somewhat remote from rail transportation lines, yet not so far from sources of raw material or from markets but that with improved highways and improved motor vehicles the highways may be profitably used for hauling their products. Such industries do and will tend to increase highway traffic.

4.—Opportunities exist for industrial and farming development in innumerable localities remote from transportation lines. Highway transportation will lead to development of the resources of these localities and, in turn, will re-act to increase further highway traffic.

5.—Passenger transportation by means of busses is increasing rapidly. Already there are several bus lines making regular daily trips between cities 100 miles apart, and many with hourly schedules between nearer points.

6.—There is always the inherent desire of the dwellers in congested areas to reach and enjoy the open country and for dwellers in the country to reach and enjoy the city scenes and activities. The average person enjoys a trip anywhere away from his own usual environment; there is also the professional man, the business man, and the artisan, who enjoy using, or may frequently of necessity use, the motor vehicle in preference to the old form of public or private conveyance. All these are large factors tending to increase highway traffic.

All these elements affect the development of highway traffic, but in order that such development may continue in healthy progress, the nations and subdivisions thereof must enact uniform and consistent laws and ordinances of permissions and prohibitions, particularly as related to dimensions and speed of vehicles.

VAN ALEN HARRIS,* M. AM. SOC. C. E.—The viaducts across Biscayne Bay between Miami and Miami Beach, Fla., are illustrations of traffic saturation. The distance across the Bay is three miles, of which the larger part is a causeway with plenty of room for a very dense traffic. At each end of the causeway is a viaduct 2 000 ft. long and 20 ft. wide between curbs, allowing only one line of traffic in each direction. The speed limit is 25 miles per hour, with no passing of cars going in the same direction on the viaducts.

Miami Beach, with a building program of $4 000 000 per year, is a high-class residential section, and also the ocean bathing resort of Miami. Practically all labor working at Miami Beach comes from Miami across these viaducts and causeway. This flow of workmen returns to Miami between 4:00 and 6:00 P. M., which is the same time that the bathers return to Miami, with the consequent peak traffic between these hours.

In the summer of 1924, that is, out of the tourist season, 8 400 automobiles and trucks were counted crossing in 1 day, and between 1 200 and 1 300 in 1 hour, at the peak of the traffic, of which about 85% was going in one direction. No more could have passed, as there was a steady stream of cars which were urged to go as fast as possible, but, although the speed limit is 25 miles per hour, the average speed during the congestion was about 18 miles per hour. It may be assumed, therefore, that about 1 100 cars in each direction is the limit of a 20-ft. road under the most favorable conditions.

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THE EFFECT OF MOISTURE ON CONCRETE

Discussion*

By A. T. Goldbeck, Assoc. M. Am. Soc. C. E.

A. T. Goldbeck,† Assoc. M. Am. Soc. C. E. (by letter).‡—Those having to do with the design of concrete structures are likely to consider the safety of the structure from the standpoint of its ability to carry loads. It has been the experience, however, that the life of many structures has been limited by factors other than their ability to carry the loads for which they have been designed. It is recognized that concrete highways are subjected to stresses of considerable intensity, which are produced by agencies other than the wheel loads of traffic, and a thoroughly rational procedure in the design of concrete pavements must give due weight to all the high stress producing possibilities. It is known, of course, that temperature change brings about high stress in concrete pavements, and it is also quite apparent that moisture is another very active stress producing agent.

The present investigation was made to obtain more exact information on the influence of moisture on concrete highway design. The author is to be commended on the careful and painstaking manner in which the present series of tests were conducted. The careful design of apparatus and attention to necessary detail in the conduct of the work are reflected in the character of the results obtained. The author has corroborated the general results obtained by other investigators in this field, namely, that Portland cement mortar or concrete shrinks on drying and expands when kept moist. It is significant, however, that the percentage of contraction on drying differs greatly in different brands of cement. Obviously, this must be an important element in the relative number of shrinkage cracks in concrete roads after the initial period of curing. It is desirable, of course, that the cement should shrink as little as possible. The variable coefficient of expansion with temperature shown by these tests is a fact that has not generally been recognized, for it is quite the custom to use a constant value for the coefficient of expansion irrespective of the temperature. Apparently, the value for the coefficient of expansion at ordinary air temperatures is about 0.00000425, which is somewhat lower than the value ordinarily accepted.

The values obtained by Professor Hatt might be well worth while corroborating through another series of investigations. Values obtained for a change in length of $1 : 2 : 3\frac{1}{2}$ concrete in general check with those obtained by

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the writer.* The decrease of contraction depends apparently not only on the character of the cement, but it has been the writer's experience that much also depends on the dryness of the atmosphere, for it has seemed that even a short period of high humidity could be detected through the slight expansion of a very dry specimen.

It is to be noted that the percentage of expansion of concrete kept moist is considerably lower than the percentage of contraction of concrete permitted to dry out. This conclusion likewise was borne out in the writer's experiments along this line. The effect of a period of wet weather on specimens exposed to the atmosphere is likewise clearly demonstrated. It is to be noted, however, that the rate of change of length of the specimen is quite slow, not only in expansion, but also in contraction. It has often been noticed that the greatest percentage of "blow-ups" in concrete pavements occurs in the spring and that as the summer progresses the pavement settles back into position. This phenomenon is readily explained by the tests just reported by Professor Hatt, for in the spring the sub-grade is in its wettest condition. The concrete, therefore, is expanded not only by moisture, but by the high temperatures during that period. Gradually, the moisture evaporates and during the summer the concrete shrinks somewhat and is subjected then only to the expanding action of high temperature.

The lesson to be learned from Fig. 10† evidently is that engineers need not be afraid of the structural strength of concrete as long as it is in a frozen condition, but that its weakest condition is that immediately after thawing or when it is saturated. The measurements on the movement of the corner of the road slab, depending on the extent of saturation from bottom to top, are what might be expected, and the deformations calculated from these deflections agree strikingly well with the measured deformations.

Perhaps it is not generally realized that under unfavorable conditions a rather high stress might be created transversely in a concrete slab due to a condition of high moisture at the bottom of the slab as compared with a dry condition at the top. For an 18-ft. slab, the following calculation shows that theoretically a stress produced by deflecting the end of an 18 ft. by 1 ft. by 8-in. concrete beam to its original shape after it had been warped because of difference of moisture content between top and bottom, may approximate the modulus of rupture of concrete.

From Fig. 24 (a):

$$R = \frac{C^2}{8\,m}; \quad R = \frac{d \times L}{d\,L}; \quad C = L; \quad d\,L = c'\,L$$

$c_1 =$ Unit change in length between top and bottom fibers $= 0.00025$.

By substitution:

$$m = \frac{c' \times L^2}{8\,d} = \frac{0.00025 \times 216 \times 216}{8 \times 8} = 0.182 \text{ in.}$$

Assuming the beam to be supported at the center, each half becomes a cantilever of length $= 108$ in. (Fig. 24 (b)).

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* *Bulletin No. 532*, U. S. Dept. of Agriculture.
† *Proceedings*, Am. Soc. C. E., May, 1925, Papers and Discussions, p. 769.

The deflection due to weight of beam is:

$$f = \frac{W\,l^3}{8\,E\,I} = \frac{900 \times 108 \times 108 \times 108 \times 12}{8 \times 3\,000\,000 \times 8 \times 8 \times 8 \times 12}$$
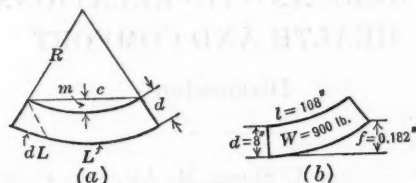
$$f = 0.094 \text{ in.}$$

FIG. 24.

The stress produced by weight of beam is:

$$\frac{W\,l}{2} = \frac{S\,I}{C}\;;\quad S = \frac{W\,l\,c}{2\,I}\;;\quad S = 380 \text{ lb. per sq. in.}$$

The remaining deflection of 0.088 in. will be accompanied by a stress of 271 lb. per sq. in., and was found from the formula:

$$S = \frac{3\,f\,E\,d}{2\,l^2}$$

The total stress will be:

$$380 + 271 = 0.651 \text{ lb. per sq. in.}$$

This calculation assumes that the slab is bent back into a plane by passing vehicles after it has been warped by the unequal distribution of moisture from the bottom to the top of the slab. This condition is, of course, an extreme one and is generally not obtained. Nevertheless, the possibility of very high stress is quite apparent. It should be borne in mind that, in general, the extremes of moisture which can be obtained under laboratory conditions are not likely to occur in the field, for usually the evaporation and saturation are not carried to completion in the field, due to the intervening influence of periods of dry and hot weather and to the influence of capillary moisture from the sub-grade.

The study of the effect of moisture on concrete would not be complete without further study of the effect of repeated loading on wet concrete as well as on dry concrete. Professor Hatt and others have shown that repeated loads have telling effect when they produce a stress much in excess of the modulus of rupture of the concrete, the concrete being dry. It would be interesting to learn whether the effect of repeated loads might not be more severe on concrete in a saturated condition.

THE ATMOSPHERE AND ITS RELATIONS TO HUMAN HEALTH AND COMFORT

Discussion*

By George A. Soper, M. Am. Soc. C. E.

George A. Soper,† M. Am. Soc. C. E.—The speaker feels that he has been on a long journey. From the stellar spaces to the bowels of the earth, from the abrasives factories of Niagara Falls to the fogbound cities of Europe, from the Gobi Desert of Mongolia to the hermetically sealed testing chamber of a New York university, is certainly a great distance. He hopes he has escaped from all the dangers of breathing air which Professor Winslow has told about but he is not sure.

Carbon monoxide is a subtle poison, and since city dwellers are compelled to breathe it day after day and year after year, there ought to be more certainty as to its effects. The danger, if it is one, is increasing. It is increasing with the size and complexity of urban life. With the growth of traffic and of industry, the air is receiving more and more carbon monoxide every day and, with the greater height of buildings, the opportunities for its diffusion and removal are becoming less.

One of the principal sources of carbon monoxide may be mentioned, to show how rapidly the increase in this gas is taking place in the city atmosphere. In 1923, the number of motor cars registered in New York, N. Y., was 363 590. In 1924, the number had grown to 440 218. The increase was 21% in this one year. The volume of gasoline consumed by these cars in 1924 has been estimated at 185 181 000 gal.

It may easily be that many persons are suffering from slow poisoning and that effects of which there is as yet no proof are being produced that are decidedly injurious. It is a curious fact that people so seldom attribute to the air the sensations of discomfort, if not of ill health, which they experience from it. They sit in crowded and ill-ventilated rooms, until they get tired, little reasoning that it is the state of the atmosphere which fatigues them.

There can be no disagreement as to the significance of carbon monoxide as a cause of sudden death, when present in sufficient concentration. Dr. Charles Norris, Chief Medical Examiner of the City of New York, in an address read March 25, 1924, before the New York Academy of Medicine, and

again before the Annual Meeting of the Police and Fire Surgeons of the City of New York, said:

"Carbon monoxide poisoning is of immense interest to the community, for the reason that it claims yearly, in all modern cities, more deaths than occur from any other one cause (including accidental and suicidal gas asphyxiation), except perhaps, in New York City, the deaths from vehicular accidents."

The speaker has said that people are slow to ascribe to the air about them the discomfort which they feel from it. This is particularly true of carbon monoxide. A person may fall over, gassed to unconsciousness, without ever having realized that anything was the matter with him. He may sit and breathe air which is contaminated with this gas and not be made in the least uncomfortable by it until he starts to walk home in the outer air when the symptoms may attack him with such violence that he is unable to proceed. A motor idling in a closed garage for 20 min. will produce so much carbon monoxide that a person who is working on the car may suddenly fall, lose his senses and die without ever crying out. Many deaths occur from gas heaters and stoves, which give off carbon monoxide to the atmosphere of sleeping-rooms, the lives of the sleepers passing quietly away without the victims waking up.

Some time ago a family was asphyxiated on the eighth floor of a house from a leak in a pipe in the cellar. Recently there were two deaths in an apartment house in New York ascribed to a small leak in a main in the street. The speaker has broken down a door and dragged out a man near the point of death from a bathroom in which a gas water heater had produced enough carbon monoxide to overcome him. In none of these instances has there been any reason to suppose that the victims have struggled or made any effort to get better air. The gas has robbed them of the power of doing so.

So much for the poisonings of a tragic character which are due to carbon monoxide. Engineers are not unfamiliar with them, for they occur occasionally in mines, tunnels, and some other works of engineering construction. When these accidents are spoken of, however, the tale is not all told. It is necessary to consider the large amount of poisoning from this gas which does not lead to fatal consequences but produces effects which may nevertheless be harmful. Just what these effects are and how harmful they may be is a question which should be considered.

Carbon monoxide is a practically odorless, tasteless, invisible gas, of almost the same specific gravity as pure atmospheric air. It is discharged from automobiles in large quantities, owing to the incomplete combustion of their gasoline.

Are the poisonous gases diluted sufficiently under the everyday conditions of city life? In answering this question two conditions must be taken account of: First, the state of the general atmosphere of the streets and other places where the gases are encountered, in which diffusion is fairly uniform and complete; and, second, the air close to an automobile before the exhaust gas has had time to become thoroughly mixed with the air about it. All know from experience how unpleasant it is to get a breath of the odorous gases direct from an exhaust. One instinctively stops breathing for an instant in order to avoid

it. The concentration in this second case is many times that of the first and is hazardous in the extreme.

The standard adopted for the New York-New Jersey Vehicular Tunnel is 4 parts per 10 000, it being expected that this standard will apply only to passengers making the transit of the tunnel in less than 1 hour and usually in less than ½ hour. Employees, who are exposed longer, are to be provided with an extra amount of ventilation at their stations.

A word as to the way in which carbon monoxide does injury. As every one knows, oxygen is so necessary to life that to be deprived of it causes instant death. It is absorbed from the atmosphere by the hemoglobin of the blood as it passes through the lungs. Hemoglobin will absorb carbon monoxide also. It has a decided preference for it. It takes up the carbon monoxide three hundred times more readily than it absorbs oxygen. It accumulates it and keeps it for a long time. A person recovering from carbon monoxide poisoning gets well very slowly. The gas is not irritating or corrosive. It is obstructive. It keeps the blood from absorbing the amount of oxygen which the person needs.

The intensity of the effects depends not only upon the amount of carbon monoxide in the atmosphere but also upon the length of time that one is exposed to it and upon various other conditions. One person may be much more quickly affected than another. In mine accidents where a number of men are killed, others may extricate themselves from the group and walk away. Long exposure to low concentrations produces worse effects than short exposure to high concentrations, even though the total absorption is the same.

Increased breathing facilitates the absorption, so that walking and other exercise accelerates it. Anything that lowers the physical fitness of a person, as fatigue and overwork, contributes to the harmful effects. Children and persons in ill health are ordinarily the first to be affected.

For the blood to take 60 to 80% of the greatest amount of carbon monoxide which it is capable of absorbing is fatal. To absorb 30 to 40% is to produce severe headache, weakness, dizziness, dimness of vision, nausea, and collapse. To absorb 10 to 20% produces a sense of tightness across the forehead, perhaps a slight headache.

When the concentration of carbon monoxide in the air is from 2 to 3 parts per 10 000, the blood will absorb from 23 to 30% in from 5 to 6 hours. When the concentration is 4 to 6, the percentage becomes 36 to 44 in 4 to 5 hours. When the concentration is 7 to 10, the percentage rises to 47 to 53 in 3 to 4 hours.

In other words, if a person breathes air containing between 2 and 3 parts of carbon monoxide per 10 000 for 5 or 6 hours, he is very likely to have a headache with throbbing of the temples, and long before that he may feel a tightness across the forehead and a slight headache, and the blood vessels of his skin may be somewhat dilated.

Now as to the concentrations of carbon monoxide which are to be met with in city streets.

In 1923, Professors Henderson and Haggard of Yale were employed by the Committee on Public Health of the New York Academy of Medicine to make

tests of the amount of carbon monoxide in the air of the streets of New York. This they did, between the middle of July and the following March. The samples were collected into suitable containers, which were carried about in a motor car along the streets.

The investigators showed an inclination, as many visitors do, to remain on Fifth Avenue. They went repeatedly up and down that thoroughfare, once as far north as 135th Street, although it is to be noted that, from 59th to 110th Streets, one side of the Avenue is entirely open to Central Park and the air is better for this reason. Following are some of the results of the analyses. One July afternoon, with light traffic, the carbon monoxide among fourteen samples ranged between 0.1 and 0.64 parts per 10 000. On a rainy day, with light traffic, sixteen samples were taken and had a range between 0.5 and 2.6. On a winter morning, with moderate traffic, twenty-seven samples ranged between 0.2 and 4.6. A series of nineteen samples, with considerable traffic, ranged between 0.2 and 3.1. On still another day, with moderate traffic, fifteen samples varied between 0.2 and 2.9. The average of these ninety-one samples of air from Fifth Avenue was 1.1 parts of carbon monoxide per 10 000 parts by volume of air.

In various parts of the city, on a clear, cool day with light traffic, samples were collected gradually over a period of 20 min. and ranged between 0.6 and 2.1. Finally, on a very cold, gusty day, seven samples were collected which ranged between 0.65 and 1.5.

The tests were made on samples of air collected at times of moderate traffic and, for the most part, on a broad and open thoroughfare under conditions which may be said to have been, on the whole, favorable to low carbon monoxide concentrations. On every day except the first one, when the traffic was light, the samples contained more carbon monoxide than is safe and some of them so much as to be positively alarming.

The investigations should be repeated on a larger scale, now that the number of automobiles has increased so greatly, and they should include a greater variety of places, times, and concentrations of traffic. Certainly, they should cover times when the traffic is heavy, for this is a very common condition and one to which particular interest attaches. Not only is more carbon monoxide produced then but there are more people in cars and on foot to be affected by it.

Not only the maximum and minimum and average condition of the air but the worst conditions which frequently occur should be known. For example, Sixth Avenue under the elevated railroad in the lower Forties is often filled from curb to curb with cars, either moving slowly or idling. Overhead is the low-lying structure of the elevated railroad to interfere with the diffusion of the exhaust gas to the upper atmosphere.

How much carbon monoxide is present when the avenues are blocked with traffic? How much is present when the blue, oily smoke from the motor cars hangs, a thick haze in the streets and there is no wind to blow it away? What is the concentration when traffic is jammed on the narrow cross streets? What is at the taxi platforms of the railroad stations, when it seems, as it usually does, so stifling?

The information collected by a thorough investigation would apply not only to people temporarily on the streets, important as that would be, but it would have more far-reaching value. It is not to be forgotten that the subways get their air from the streets. In fact, they get it from the surface of the pavement, which is the level at which the automobiles exhaust their gases.

The information would have relation to those who work, many of them for long hours, in the shops on each side of the streets, to the shoppers, and to the large numbers of people on the floors above the street level in the buildings which line the streets. For the shops and the buildings above them get much of their air from the streets and at the level of the front doors and cellar openings. Except in summer and under unusual circumstances at other seasons, the air which gets into the buildings enters, not so much through the windows, as does the light, but at the ground floors and cellar levels. It is heated and rises upward. Studies of the ventilation of tall office buildings of Manhattan have provided a surprising amount of interesting information upon this subject. As Professor Winslow has pointed out,* the proper ventilation of schools and shops and apartment houses and, in fact, all buildings, is not to be accomplished by fans and ducts and dampers, so much as through a better utilization of the natural currents of air which exist or may be brought about under the conditions of actual occupancy. In buildings, more attention must be given to what is called, out of doors, meteorology, and out of doors more attention must be given to what, indoors, is called ventilation. If there is to be good air indoors, there must be good air outdoors. It is of paramount importance that cities should be well ventilated.

In connection with the investigation of the air of streets, a study should be made of the health of the people who spend their working hours there. Traffic officers and the motormen and conductors of busses and trolley cars, although obviously selected as much for their rugged health as for the possession of mental qualities, would afford a profitable field for a carefully made and thorough physiological investigation.

Such an inquiry as is proposed would not be without precedent. Studies of subway air and ventilation were made some years ago at the instance of the Rapid Transit Commission of New York, in which many thousand analyses were made and these were supplemented by thorough examinations of the respiratory apparatus of the motormen and conductors on the trains.

Is the speaker exaggerating the possibility of danger from carbon monoxide in city streets? It is possible that he is. It may be that most people can stand a certain amount of carbon monoxide without much harm, just as they can stand tobacco smoke and, in former times, some stood alcoholic intoxication. Perhaps the only bad effects of carbon monoxide in the city is to make people tire more easily, a little less correct in judgment, a trifle less certain in mental and bodily co-ordinations, somewhat more irritable, and a bit more susceptible to accidents and disease. In the aggregate this may or may not be important. It has been suggested that some of the very numerous accidents which occur in the streets may be ascribable to the subtle effects of unsuspected carbon monoxide, but nobody really knows.

* *Proceedings.* Am. Soc. C. E., May, 1925, Papers and Discussions, p. 797.

A word as to the remedy, if a remedy is required. The speaker thinks the remedy should be left to the Engineering Profession. It is really an engineering question how to make better use of the gasoline and thus reduce the amount of carbon monoxide to a minimum. It is an engineering question how to discharge the spent gases so that they will not be driven into the faces and so into the lungs of drivers of other cars and pedestrians. It has been suggested that motor cars carry stacks and send their exhausts upward, as do locomotives. It has been proposed that absorbing chambers, filled with suitable chemicals, be carried by automobiles, for the purification of the gases which they produce.

It is to be hoped that something better than any of these solutions may be brought forward, for, aside from the question of life and health, an improvement over present conditions would certainly be beneficial from the standpoint of reasonable comfort.

LAWS RELATING TO OBNOXIOUS ODORS

Discussion*

By Joseph A. Warren, Esq.

Joseph A. Warren,† Esq.—Except in England and the United States the system of law prevailing among all the civilized Caucasian races is what is known as the Civil Law. That law takes general abstract principles and applies them to each state of facts as it comes up. It makes for rigidity, and it makes for certainty.

The American system of jurisprudence, however, is known as the Common Law, and that takes a given state of facts as it happens and applies to the interpretation of such facts the custom and the general understanding during past and present times of the community in general. As can readily be seen, under the latter system the law is continually in a state of flux, and it is very difficult to say in advance what the Courts are going to do in regard to any given state of facts. A very striking example of that is had in this very law of nuisance.

As stated by Mr. Goldsmith,‡ in the early cases the law looked at the injury to property almost exclusively. It was the damage to an individual's property that laid the foundation of all action, and that was true not only with respect to the law of nuisance, but generally. The old common law looked at the damage to property—damage to an individual suitor. With the change of the customs of the people, with the growing civic consciousness, there has been a gradual getting away from the emphasis formerly placed on property damage, and to-day the Courts look more to the interests of the community at large than to those of the individual property owner; and if the individual property owner's rights interfere or injure the community at large, the Courts are very active nowadays in finding some way of serving the general interest even at the expense of the individual property owner.

The most striking example of that attitude in recent years is the upholding of the rent laws of New York by the Court of Appeals of that State and by the Supreme Court of the United States. Formerly, the emphasis was on the injury to property, but now most emphasis is placed on the health and interest of the public or of any considerable part of the public.

In the same category is the rule mentioned by Mr. Goldsmith that one cannot move into a nuisance. At common law, if a business that might be a

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nuisance existed, and any individual who, after the establishment of that nuisance, moved into the zone of influence knowing it was there, had no remedy. Where emphasis was on the property right, this was only fair; but that law is almost extinct to-day.

Where a nuisance exists, because a community has grown up around it, the law, not to protect any individual who has moved there knowingly, but to protect the community, will make available the remedies that exist in the case of nuisance.

There was a rather striking example of the way this law works in the Edgewater case, referred to by Mr. Goldsmith. It is true that New York State in 1916 started suit in the United States Supreme Court against certain companies in the State of New Jersey; but it is not quite accurate to state that the settlement that has resulted came from the litigation. Consider the peculiar problem there. One of the finest residential sections of the great State of New York was directly opposite those factories. Some of the factories were there before the upper part of Riverside Drive had developed into a residential section. All these factories were engaged in a kind of business that gave forth fumes, odors, and smoke. They were in another State. Jurisdiction of them could not be obtained in the State of New York. They were large commercial enterprises—the Corn Products Company, the General Chemical Company, the Barrett Manufacturing Company, and others of almost equal magnitude.

Officials started dealing with the problem in 1908. Nothing very definite was accomplished until 1917, when the Legislature of the State of New York passed an Act (Chapter 292 of the Laws of 1917) that gave to the State Commissioner of Health power to investigate those alleged nuisances and, if he found nuisances that affected the life, health, and safety of the people of the State of New York, to file in the office of the Secretary of State a certificate to that effect. If within thirty days those nuisances were not abolished, the charters of those corporations in the State of New York, if they were New York corporations, or their certificates to do business in the State of New York, if they were foreign corporations, would be automatically revoked, and receivers appointed for their property. In other words, a sentence of capital punishment was imposed on those corporations for maintaining nuisances. It would have meant the destruction of certain of those great corporations.

The remarkable thing about that legislation is that no one raised the question of its constitutionality, drastic as it was. Under the direction of the then Commissioner of Health, Dr. Biggs, elaborate investigations were conducted in the first instance to ascertain the source of the particular trouble in the City of New York, and then remedies were sought and suggested. Over a period of two years the hearings went on, and a solution was gradually worked out, so that, to-day, with a few occasional and rare exceptions, that nuisance in the City of New York is practically abolished.

In this proceeding the so-called psychological effect of nuisances was given much consideration. Probably no one went further in holding what might affect the health of the people than did Dr. Biggs in that particular case. The matter was adjusted so that the Courts have never had occasion to pass

on Dr. Biggs's ruling, but without doubt they would have sustained him, particularly on the evidence in that case. It was held unnecessary to prove, for instance, that a considerable number got sore throats, or got any particular disease; but if the odors, the fumes, so disturbed residents that they lost sleep and that their general health and vitality was lowered because of this disturbance, this alone was sufficient ground to exercise the health authority of the State.

This whole case well illustrates the serious propositions that will come before sanitary engineers as the law develops in the future. In the first place, before this bill was passed various remedies had existed for years. There was the remedy by abatement in the Courts; the remedy by injunction in the Courts; action for damages in the Courts; and the machinery of the Health Department for the direct suppression of nuisance without resort to the Courts. It is thought that in the future the remedies will be modeled on the last method, that is, abatement of nuisances through the administrative powers of the State rather than through the judiciary.

The Edgewater case further illustrated the fact that, when the present remedies are not adequate, either the legislatures or the Courts are going to devise some method that will be adequate to control the situation; and the tendency in all cases nowadays is to assign the remedy to the health authorities. Now, health authorities are rather uncertain; some of them are rather extreme. One can imagine that a very serious situation would have arisen in the Edgewater case if Dr. Biggs had been an arbitrary and radical Health Commissioner who at once, without proper hearing and consideration, would have endeavored to solve the difficulty by applying the very drastic measures of that law.

It is well to remember this—when the legislature refers to a health authority the determination of a question of fact as to whether a certain business does constitute a nuisance injurious to the health of the people, the determination by that health official will hardly ever be overturned by the Courts.

This experience suggests, therefore, that when any considerable commercial enterprise is being considered, not only the immediate vicinity of the location of the plant should be carefully investigated, but the whole probable zone which might be affected by any of the processes that are proposed to be conducted.

Further, in view of the fact that a future development may make a nuisance out of what previously was not a nuisance, it seems that, in so far as is reasonably possible, an investigation should be made as to the probable development of the locality, because, once a plant is established and developed, a situation may easily arise where the whole plant would have to be torn down and moved to some other location, or where the State might have to do it.

METHODS FOR DETERMINING THE ORIGIN, PREVALENCE, AND EFFECT OF OBNOXIOUS ODORS AND THE EVALUATION OF AN ODOR NUISANCE

Discussion*

By Messrs. Charles A. Holmquist, George H. Shaw, and L. L. Tribus

Charles A. Holmquist,† Esq.—In presenting this paper the author has rendered a valuable service to the Sanitary Engineering Profession, more particularly to the Engineering Divisions of State Departments of Health which are perhaps called upon to deal with problems of this kind more often than any other organization. As indicated by the author, nuisances created by odors are among the most difficult problems with which health departments have to deal, owing largely to the intangible and elusive nature of the subject and to the lack of scientific or standard means of measuring quantitatively the magnitude of such nuisances. The detailed description of the procedure and methods used in the investigation conducted by the author of the alleged nuisance created by certain oil refineries in the Providence Metropolitan District should be very helpful to those who may be called upon to carry on similar investigations. The development of the formulas to determine the magnitude and degree of a nuisance is a long step in the right direction. The value of such formulas, however, would be considerably increased if numerical values could be given to intensities of odors similar to those given to tastes and odors of water supplies that are now in general use by water analysts. The speaker realizes, of course, the difficulty of doing this. In the first place there is no such thing as a standard nose. No two persons would probably re-act in exactly the same way to the same odor due not only to the physiological but also to the psychological difference of individuals. In fact, an odor that is objectionable to one person may be unobjectionable and even pleasing to another. Furthermore, all so-called odors are not true odors and do not stimulate the same sets of nerves. True odors, such as artificial musk and oil of roses, affect the cells of the olfactory nerves in the nasal cavity, whereas sulfurous acid and similar substances have no true odor, but are irritants which affect the ends of the trigeminal nerves. Other so-called odors affect both sets of nerves.

In view of the rapid increase in the number of industrial developments in centers of population, the ever-increasing number of sewage, garbage, and waste disposal plants in this country which are likely to give rise to nuisances

due to the emanation of objectionable odors and the growing demand for higher standards of sanitation, it is evident that this problem will have to be given more attention in the future than in the past and that this subject might well be worthy of study by a committee. A great many tests have already been made to determine the concentration of different chemicals and other substances that are detectable in given quantities of air, and their relative intensities, but additional studies would be necessary to give definite values to such odors.

The author points out that he has dealt with the methods of determining the cause, scope, and effect of odor nuisances and has left the discussion of methods of odor elimination to others. Considering the large financial investments and the great number of persons that are usually involved, remedial phases of the problem in most cases will be very important. The speaker will endeavor to discuss these phases of the problem from the point of view of State health departments and will cite a few instances that have come under his personal observation where odor nuisances have been successfully abated. In fact, it has been the experience of the New York State Department of Health that there are relatively few odor nuisances that cannot be abated by changes in operation of the plant, or the treatment, utilization, and recovery of waste products. Very often the abatement of such nuisances results in increased financial returns and sometimes in more sanitary and healthful conditions in the works themselves.

It has been the consistent policy of the Department in dealing with odor nuisances to co-operate as far as possible with the industrial plants in abating such nuisances and to issue orders and start Court proceedings only as a last resort. This spirit of co-operation is usually appreciated by the manufacturers and meets with their hearty response. Recommendations for improvements have often been carried out with relatively little delay. The owners of many of the plants investigated have probably felt, and some have admitted, that it would be cheaper in the long run to make voluntarily the improvements or changes suggested by the Department rather than to take the question to the Courts with the possibility that the improvements would ultimately have to be made at their direction. The following cases are some of the odor nuisances that have been abated through the efforts of the State Health Department without the necessity of issuing orders or bringing action in the Courts.

Carbon Disulfide Works.—These works were situated in a narrow valley extending in a generally northerly and southerly direction. Complaints were received from residents of the valley that whenever the wind was from the direction of the chemical works very objectionable and stifling odors, fumes, or gases, were noted and that the fumes discolored paint on houses more than a mile from the works. From the investigation, it was found that the trouble was caused almost entirely by the liberation of hydrogen sulfide (H_2S) in the process of manufacture of carbon disulfide and that houses in exposed areas within $1\frac{1}{2}$ miles of the works were discolored, especially those painted with lead paints. The carbon disulfide (CS_2) was made in the usual manner by passing sulfur vapors through charcoal heated to a cherry red in iron retorts. The uncombined sulfur vapors were conveyed to iron vessels in which these vapors

were condensed and the sulfur was recovered. Some sulfur dioxide also escaped when the retorts were cleaned, but this gas did not appear to be an appreciable factor in the creation of the nuisance which was caused almost entirely by the liberation of hydrogen sulfide.

When the plant was first visited an attempt was made to eliminate this gas by passing it through a tank into which lime-water was pumped. Although some of the hydrogen sulfide was probably eliminated by absorption by the water and by reaction with the lime in the production of calcium hydro-sulfite ($\mathrm{Ca\ H_2\ O_2 + 2H_2\ S = Ca\ H_2\ S_2 + 2H_2O}$), this method was not effective in abating the nuisance and was abandoned. Another attempt was made to eliminate the hydrogen sulfide by passing it through a wooden tower packed with lime and iron oxide, but the tower was found to be too small and the heat of reaction developed was sufficiently high to set the tower on fire. Finally, the nuisance was abated by passing the hydrogen sulfide through a Claus kiln consisting of two condensing chambers and an exit tower operated in series. The reaction chamber contained a layer of iron oxide through which the hydrogen sulfide mixed with a definite quantity of air, was passed after the oxide had first been heated to a dull red. This caused the reaction, $\mathrm{H_2S + O = H_2O + S}$, to take place. The sulfur vapors were condensed in the second chamber of the kiln as liquid sulfur and in the third chamber as flower of sulfur. The water passed off through the exit of the third chamber in the form of steam. Some sulfur dioxide was carried over with it, but not in sufficient quantities to give rise to a nuisance. This installation resulted not only in the abatement of the nuisance, but also in the recovery of considerable sulfur which had formerly gone to waste.

Acetic Acid Plant.—Acetic acid was manufactured at this plant by distilling grey acetated lime with sulfuric acid. A pungent acetic acid odor sufficiently strong to cause a very uncomfortable irritating sensation in the nose and throat was found in the building in which the acid was made, and this odor was carried a considerable distance from the plant. A similar pungent pyroligneous odor resembling acetic acid was also given off from the residue from the acetic acid stills and this odor seemed to carry very much farther than the acetic acid odor. When this plant was first inspected it was customary for the company to dump the residue from the stills on the floor of the still house, shovel the residue into dump carts, and haul it to a dump on the water-front where it would be left until a barge load or two had accumulated. This dump seemed to be the principal source of the odors.

The nuisance was abated by collecting and scrubbing the air in the building in which the acetic acid stills were located, by cleaning up the water-front, and by installing mechanical conveyors by which the residue from the stills was conveyed through covered chutes to covered gondola cars on a railroad siding outside the plant. The cars were removed as soon as they were filled. The scrubber through which the air from the building was passed consisted of a wooden tub about 10 ft. in diameter and 10 ft. high, filled with coke. Numerous sprays of water played continually on the surface of the coke, and the exit from the scrubber was connected to the stack of the power-house plant which was about 200 ft. high. Although no waste products were re-

covered, there was an appreciable saving in the cost of handling the residue by the mechanical conveyor.

Sulfuric Acid Plant.—In one plant of this kind against which complaint was made, the sulfuric acid was manufactured from pyrites by the contact process. Objectionable conditions were created by the escape of sulfur dioxide and a reddish dust from the burned pyrites which was stored on a dump outside the plant. Sulfur dioxide was found to be escaping from the exits of the absorbing towers and was so strong as to be almost unbearable within a few hundred feet of the plant when in line with the wind from the exits. The nuisance from this source was abated by utilizing the waste sulfur dioxide in the manufacture of sodium bi-sulfite. The exits through which the uncombined sulfur dioxide from the sulfuric acid plant formerly discharged into the atmosphere were closed, and the gas was conveyed to an adjoining building where it was passed through absorption towers through which a concentrated solution of sodium carbonate was circulated which resulted in the production of sodium bi-sulfite. About 80% of the sulfur dioxide was absorbed in these towers and a large part of the remainder was removed by scrubbing the exit gases from the bi-sulfite plant with water. The dust nuisance from the pyrites dump was eliminated by spraying the dump with water. Crude sulfur was later substituted for the pyrites in the process of making sulfuric acid.

At another plant where sulfuric acid was made by the chamber process, the acid was concentrated in platinum stills. The gases and acid escaping from the stills carried for a considerable distance and gave rise to a serious nuisance. These conditions were abated by changing the method of concentration and by installing a Cottrell machine for the condensation of the acid fumes.

Linseed Oil Works.—Characteristic odors given off from the process of making linseed oil although very pronounced near the works did not seem to carry any considerable distance from them. Very pungent odors, however, were given off from the process of boiling or aging the oil. This was done in covered steam-jacketed kettles in which the oil was heated to a temperature of about 350° Fahr. for several hours while air was blown through the oil. When the oil works were first visited the oil vapors from the boiling kettles were conveyed to the breaching of the boiler plant. Owing, however, to the relatively low temperature at this point, and probably also to deficiency in oxygen, the oil vapors were not consumed but were simply scorched. Although the resulting odors could not be detected on the premises, they were very objectionable at a distance of a mile or more from the works. This nuisance was abated by connecting the vent pipe from the kettles with the firebox of one of the boilers. This seemed to produce complete combustion and eliminate the nuisance.

Coal-Tar Products Plant.—At a plant where coal-tar products were manufactured, large volumes of yellowish pungent oil vapors were given off when the stills were blown into the coolers. This nuisance was abated simply by closing the manholes on the coolers and installing vents by which the vapors were conveyed to coil condensers.

Starch and Corn Products Plant.—In this plant the corn was cleaned and then soaked or steeped with water containing sulfurous acid for the purpose of softening the kernels. After steeping, the corn was passed through cracker

mills where it was partly crushed and the germ removed from the body of the kernel. The germ was separated from the remainder of the corn by gravity in shallow tanks filled with water and was dried in rotary driers, and ground, heated, and pressed to produce corn oil. The corn from which the germ had been removed was then ground in Burhstone mills, and the mixture was washed and passed through fine mesh screens and silt reels a number of times in order to separate the starch from the gluten and other fibrous matters. The gluten and fibrous matters were passed through mechanical squeezers to remove the water, after which they were dried in rotary steam-heated driers to form gluten feed. The starch was settled out in long inclined troughs and then dried. Glucose was made by boiling starch paste with mineral acid, neutralizing the acid and evaporating the liquid.

A number of gases and odors were given off from the process of manufacture, including sulfur dioxide and sulfurous acid from the sulfur burners and the sulfurous acid plant and from the screen and reel rooms. A peculiar odor was also given off when the contents of the glucose retorts were blown into the coolers. The odor that seemed to cause the most trouble, however, was that discharged from the rotary driers where the gluten feed and the germ were dried. There were twenty-two of these driers which were vented into eleven large wooden stacks. Vapors approximating 40 000 cu. ft. per min., were discharged from these stacks. The odor of these vapors although difficult to describe resembled a combination of acid and organic matter not much unlike the smell of sour bread. These odors carried for a considerable distance and seemed to be nearly as intense a mile or two from the works as they were at the works. Considerable chaff was also discharged from the drier stacks with the vapors. A large part of this chaff was deposited on the adjoining buildings and premises, and some of it was carried long distances by the wind.

It was first thought that these odors could be eliminated by collecting them and passing them through a large water scrubber. Owing to the structural difficulties involved and the very large sized scrubber necessary to care for the vapors, tests were carried on with a new type of scrubber. This consisted of two concentric cylinders connected by a spiral plate arranged so that the vapors would take a spiral course in traveling from the inlet to the outlet while water was sprayed into the space between the two cylinders in such a way as to keep the surface against which the vapors came in contact wet. The vapors were forced through the scrubber by a fan at a velocity of about 5 000 ft. per min. While these tests were being made, an incident occurred which indicated very clearly that scrubbing the vapors by water alone would not eliminate the nuisance. One day while observations were being made at a point about a mile from the starch works, a very severe rain storm occurred. In order to determine the effect of the rain on the odors, the observer continued his observations in the rain for about 30 min., and it was found that, although the odors passed through a natural mammoth scrubber about a mile in length, they were not appreciably diminished in intensity.

An attempt was then made to deodorize the vapors by applying different volumes of chlorine gas to the base of the drier stack but, although sufficient chlorine was applied to be distinctly noticeable a considerable distance from the

plant, the odor was not reduced to any appreciable extent. This was probably due in part to the inefficient mixing of the chlorine gas with the vapors and the short time of contact, but largely to the inability of the chlorine to deodorize or oxidize the particles of chaff carried out with the vapors. Tests were then made with the application of chlorine gas in connection with the experimental scrubber. Chlorine was applied first to the inlet and then at the outlet of the scrubber. Although the application of the chlorine gas to the inlet of the scrubber seemed to have no marked effect on the odors, its application near the outlet of the scrubber, but ahead of the fan, seemed largely to eliminate the odors, especially when sufficient water was used practically to condense the vapors and remove the chaff.

The company found, however, that the scrubbing and chlorination of the large volume of vapors given off from the exits of the driers would involve a considerable expenditure and would be costly to maintain and, therefore, abandoned that project. The vapors were finally collected and after passing them through scrubbers of the type experimented with, in order to remove the chaff which would otherwise have collected in the conduit, were conveyed to the forced-draft conduit of the power plant where they were mixed with about 200 000 cu. ft. of air per min., and burned. The burning of these vapors under the fireboxes of the boilers and the installation of a condenser on the exits of the glucose blow-tubes seemed to abate the nuisance effectively. Although no by-products were recovered as the result of the abatement of this nuisance, the sanitary conditions around the plant were improved appreciably, and it is no longer necessary to employ men to clean up the chaff which formerly settled on the roofs of adjacent buildings.

Many other cases could be mentioned but a sufficient number have been cited to indicate, in general, the nature of the problems involved in abating odor nuisances. Some of the problems are very simple; others are more complicated and difficult, but most of them can be solved by the intelligent application of chemistry, engineering, and common sense.

George H. Shaw,* M. Am. Soc. C. E.—As Chief of the Division of Housing and Sanitation in the Department of Public Health of Philadelphia, Pa., the speaker has encountered conditions similar to those described by Mr. Gage† and Mr. Holmquist.‡

Philadelphia, with 2 000 000 inhabitants, is one of the largest manufacturing cities in the United States. In many places, manufacturing plants are located in close conjunction with residences; probably the conditions in this respect are as bad as those in any city in the country.

One of the serious nuisances is the manufacture of red oxide paint. In one large plant sulfurous acid is driven off and discharged from a tall stack spreading the fumes over the neighborhood. Everything possible was done to get the management to control this nuisance without success. It was

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* Philadelphia, Pa.
† *Proceedings*, Am. Soc. C. E., May, 1925, Papers and Discussions, p. 820.
‡ See p. 1169.

finally necessary to take the case to Court and secure action requiring the manufacturers to abate this nuisance, which they did by discontinuing the "roasting" of the raw material in Philadelphia and transferring that operation to another city.

Another troublesome type of plant is that used for rendering. In one case it was necessary to institute criminal proceedings. Realizing that the law was sufficient to compel it to correct this condition, the company came to terms and spent a considerable sum of money installing control apparatus to abate the nuisance. Since then there has been no trouble.

Packing house supply companies now sell rendering apparatus so constructed that the odors do not escape into the atmosphere. The apparatus is air-tight, and has an outlet to the sewer; so there is no further trouble with the cooking odors from open jacketed kettles. Many improvements in the handling of material have also been developed by packing house engineers.

Another nuisance producing industry that has given a great deal of trouble in Philadelphia is the oil refinery. In one large refining plant, sulfuric acid is used, which causes a nuisance when it is being recovered by distillation. To correct this condition a Cottrell electric precipitator was installed and has given satisfactory results.

In Philadelphia, there is a nuisance that Mr. Holmquist did not mention, that is, one due to the manufacture of electric storage batteries. Even workmen living close to such plants, who liked to sit on their porches in the evening and enjoy the fresh air, could not do so because of the presence of irritating acid fumes given off from the "forming room", where battery plates are manufactured or "formed" by electrolytic action. The co-operation of the company was secured and satisfactory control apparatus was developed. The air from the "forming rooms" is forced by fans through a condensing chamber filled with staggered fluted glass plates. The air, as it is drawn from the "forming room", goes through a tortuous passage, depositing the acid on the glass plates whence it trickles down the fluted surface and is collected at the bottom. A considerable quantity of acid is thus recovered.

It is believed that nuisances of the types mentioned have an indirect if not a direct effect on health and that their regulation properly comes within the "police power of a community." Experience has shown that they can be corrected and that rarely is it necessary to take drastic Court action. Helpful co-operation with the offending plant usually brings the desired result.

L. L. Tribus,* M. Am. Soc. C. E.—Mr. Gage has brought out one of the important features in considering odors, namely, the psychological effect—the effect that may be positively injurious and constitute actual legal nuisance. This was discussed somewhat in the speaker's paper entitled "Odors and Their Travel Habits."†

There is a peculiar difference in the effect of odors on different individuals, their personal equation. An odor might well be profoundly disturbing to the

whole nervous system of some strong man, and, at the same time, be fully tolerated by some otherwise weak woman.

Another condition might be considered. How long does an odor affect an individual? Does the effect continue after the actual atoms have become dissipated, and memory reproduces the condition? The speaker thinks that it does, that even nausea can be reproduced long after the original smelling of the odor, if the first experience had caused such a condition.

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ELIMINATION OF ODORS FROM GARBAGE
DISPOSAL WORKS
Discussion*

By Messrs. John V. Lewis and I. S. Osborn

John V. Lewis,† Esq.—This paper brings out two points of prime importance which must be apparent to every sanitary engineer who has been concerned with the problem of refuse disposal at some time or other in his career.

The first point is the need for a collective technical effort and thorough engineering study of this ever-perplexing and acute problem which almost every growing municipality now faces. With due respect to those who have "blazed the trail", it is to be regretted that the same combined effort has not been accorded the disposal of rubbish, garbage, or mixed refuse, as has been given to sewage disposal. Had that been done, it is safe to say that the status of refuse disposal in its entirety would be quite another story.

The second point—and indirectly it is really a part of the first one—is that it should be understood that the problem, particularly as regards the control of odors, is one of engineering in its broader sense, namely, the education of the public and the injection of economics, just as much as the pure and applied sciences involved.

As mention has been made by the author of the Rochester, N. Y., plant as one of the most approved design and free from complaint as to obnoxious odors, some additional information may be of interest. From 1907 to 1921, the garbage of the city was reduced in a so-called modified Arnold plant. It was first cooked in large vertical steel digesters, and the resulting mass was then passed through hydraulic presses for the recovery of the grease. Drying and subsequent percolation were not used. The pressed tankage was sold for fertilizer.

The speaker is informed that the odors emitted from the plant, situated as it was in a commercial and industrial district about ¾ mile from the business center of the city, were the cause of numerous complaints during the period of operation. Most sanitary engineers are quite familiar with the difficulties which beset any one who advocates a new location for a garbage or mixed refuse disposal works, once it has been previously established. Thus, in constructing the new Rochester Reduction Plant, which began operation in 1921, the so-called straight Cobwell system was adopted as that which would best eliminate the cause of previous complaints and enable the existing site to be

utilized.. This site, which is at the geographical center of the city, greatly influences the haulage and several other important questions. The speaker believes that the success of this new plant and the failure of a similar one on Staten Island, New York, was essentially a question of geography and economics and not a lack of understanding of the technical side of the problem itself. The City of Rochester has the most up-to-date and sanitary garbage plant of its size in the country to-day, free from objectionable odors which may be the source of formal complaint. There is a constant endeavor to exercise technical control of the process used; to reduce still further the slight odors which do exist; and to place the collection system and the plant on an economic business basis. Such an attitude and effort were intended by those responsible for the inception of the new system.

As Mr. Greeley has pointed out, the control and possible elimination of odors from garbage disposal works must begin at the garbage pail itself, continue through the collection and haulage system, and end finally in the disposal works proper. If either of the modern sanitary methods of disposal called incineration and reduction are used, every effort exerted to collect the raw material in its fresh state and with the least water content means better control of odors and a reduction in the operating costs, both as regards haulage and the method of final disposal.

When through proper education of the public and diligent co-operative engineering effort, the collection and disposal of mixed refuse or garbage can be said to approach present-day methods for the collection and treatment of sewage, the problem will be nearer to a satisfactory solution.

I. S. Osborn,* M. Am. Soc. C. E.—In his paper on the elimination of odors from garbage disposal plants, Mr. Greeley brings out the fact that each stage in the treatment has a bearing on the subject as a whole. The house treatment, methods of collection, types of equipment, and methods of disposal are so interrelated as to require consideration of each.

If the most desirable solution from a sanitary standpoint is expected all phases of the problem must be considered. The author has indicated that the location of disposal plants in some cases should be determined by the type selected, and that the choice of a plant emitting odors requires an isolated site, whereas one free from odors may be nearer the center of production. Too little attention is given to the selection of the type of plant which can be erected in a central location when other factors enter and appear to dominate or control, such as length of haul, or cost of delivery of the refuse, and possibly ownership of the site where the right to operate has been established.

Two cases of this kind have come to the speaker's attention recently. In one case, the accepted location increased the cost of haul by $30 000 per year over that of another available site, although the site selected had no advantages in its surroundings over the one rejected. In the other case, a site outside the city limits was selected and by so doing the hauling costs were increased approximately $50 000 annually over what would have obtained if another available site had been chosen. In each case the type of plant adopted was

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such that it could be operated in many locations without producing odors and without giving rise to complaint.

In many cities, the type of disposal plant is selected before a site has been chosen on which the plant can be located. However, the plant at Rochester, N. Y., mentioned by Mr. Greeley, was selected by a method directly opposite. The City owned the site and selected the type of plant to meet its surroundings. No other reduction plant is so well situated from the standpoint of cost of delivery of the garbage to the plant.

The author's statement* with reference to the location of a reduction plant in isolated places in order to lessen the necessity for odor control is hardly supported by the facts. There is not a reduction plant disposing of garbage in an isolated location, and causing odors, to which those in the community where it is located do not object.

There are, no doubt, fewer complaints due to the smaller number of people affected. In some cases the few tolerate the nuisance rather than assume the burden incident to a protest.

The establishment of districts under a zoning law, where odor-producing industries may be located, is done to protect property and industry. If plants, such as packing houses, rendering plants, and garbage disposal plants, are permitted, it is done to protect the industry as well as the public and not to legalize a nuisance from plants producing odors which may be eliminated.

With the Court rulings which have been made there is a question whether, even in an isolated location, the operation of a plant giving off odors will be permitted unless it is conducted in a manner to create as little nuisance as possible.

In discussing incinerators, the author states that "ample combustion and dust-settling chambers" should be provided, and that "satisfactory incineration over a number of years calls for durable construction with properly designed furnace structures to stand the relatively hard service."

Some of the fundamental features that should be incorporated in furnace design to give satisfactory service are, as follows.

Temperature.—The furnace should be designed so that a uniform temperature can be maintained when burning material with low heat value and high percentage of moisture, with assurance of temperature at all times sufficient to obtain complete combustion or oxidation without part of the gases passing off by distillation.

Fuel.—The design should be such that material containing a small quantity of combustible matter and a high percentage of moisture will burn without using additional fuel. This can be accomplished only by utilizing at maximum efficiency the heat developed in its operation.

Maintenance.—The cost of the maintenance of furnaces will depend on their design and the incorporation of features calculated to give proper protection under severe service. The design should permit of performing maintenance work without disturbing other parts of the furnace not in need of such work.

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* *Proceedings*, Am. Soc. C. E., May, 1925, Papers and Discussions, p. 833.

Thermal Efficiency.—The design should be such that the thermal efficiency will be high, using materials and construction that will insure only a small loss from radiation, with temperature developed to a maximum.

Draft.—The design should insure that all parts of the furnace can be balanced as to draft, including the furnace chamber, combustion chamber, flues, and chimney.

Simplicity.—A furnace for burning refuse adapted to the service for which it is intended, as well as the class of labor to be employed, and the material to be burned, should be simple in design.

Combustion.—Furnaces to insure proper combustion, should have:

(a) Ample space for combustion and the burning of material carrying a large quantity of hydro-carbon.

(b) Maintenance of steady high temperature in the fuel bed and in different parts of the furnace chamber.

(c) An adequate air supply, thoroughly distributed at the point where combustion takes place.

(d) Charging of the furnace and the manipulation of the fire without opening the fire doors or charging doors.

(e) The maintenance of a fuel bed on the grate whereby a variable quality of material will not influence the combustion or cause fluctuation in temperature.

(f) A high rate of combustion for maintenance of temperature and satisfactory operation.

Pre-Heating Forced Draft.—The pre-heating of forced draft air aids in combustion by supplying heat, thus permitting the burning of a lower grade of material; and, due to the expansion of the air in pre-heating, it permits of better distribution and, at the same time, reduces the excess air that would be required if it were not heated.

Structure.—To withstand the continued heating and cooling of the furnace, as well as to take care of the stresses developed under high temperature, the furnace should be designed as a structural unit with all the stresses that are developed in it taken up by the structural parts, the brick housing being installed for refractory and insulating purposes.

The author presents a table (Table 4)* of analysis of organic garbage with the estimated quantity of coal that would be required to supply fuel and maintain a temperature of 1 240° Fahr. At this temperature, all gases or combustible materials will oxidize and be destroyed without producing odor.

A furnace, however, operating under variable conditions, such as charging and cleaning, will not hold a uniform temperature. The usual practice in high temperature furnaces is to maintain a temperature higher than is ordinarily required in the combustion chamber so as to insure that the minimum temperature will be sufficiently high to guarantee complete combustion.

The burning of mixed refuse in incinerators will insure better results than the burning of wet organic waste, which requires a large quantity of additional fuel. It should never be necessary for any city to burn only organic wastes (kitchen garbage).

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* *Proceedings*, Am. Soc. C. E., May, 1925, Papers and Discussions, p. 840.

With a mixed collection of rubbish and garbage disposed of by incineration, in furnaces which consume the mixture at high temperature, the odors, with proper treatment in collection, storage, or combustion, will be prevented.

Investigation of plants burning organic waste (kitchen garbage) and using large quantities of additional fuel will show that there is a tendency to economize on fuel at the expense of operating with odors due to incomplete combustion.

Reverting to the question of odors, there are certain fundamental considerations which should govern the design and operation of disposal plants, varying, of course, with the system selected.

The adoption of preventive methods will prove more satisfactory than the elimination of odors after they are produced. Such preventive methods are applicable to all kinds of garbage disposal including feeding, dumping, burial, reduction, and incineration.

The author's statement that relatively little progress has been made in the development of methods for odor elimination is hardly borne out by the facts, if a study is made of the beneficial results which have been obtained. The results obtained and the development made should not be confused with the failures resulting from neglect to apply existing knowledge.

The greatest criticism of present practice is that engineers are not using the experience gained, but continue to follow obsolete methods. When the best known practice in design and operation of disposal plants irrespective of the type or method is applied, more satisfactory results will be secured and continued improvement will follow. Study will show that to dispose of garbage satisfactorily and in a sanitary manner, without offensive odors, the cost will be higher either in capital investment or operation, or both. This will apply to practically all methods of disposal.

Probably no other branch of municipal service has received so little real study to determine the results that can be obtained or the improvements that can be made. Many officials and engineers look on the problem as one to be side-stepped. In many instances the question is considered from what is assumed to be an economic standpoint, the selection of type of plant or method of disposal being based on the least possible capital outlay.

The sanitary engineering problems of municipalities have received study and improvement and gradual development have been made. When municipal officials and the Engineering Profession realize that the disposal of garbage in a sanitary manner is a function requiring study, and look on it with the same interest as they do on other municipal problems, improvement will be made.

When the problem is considered from this viewpoint plants will be constructed and operated which are free from odor, and there will be improvements in design and operation from every standpoint.

CONTROL OF ODORS
FROM SEWAGE TREATMENT PLANTS
Discussion*

By Messrs. Morris M. Cohn, C. E. Keefer, C. G. Gillespie and T. Chalkley Hatton.

Morris M. Cohn,† Esq.—The subject of the control of odors from sewage plants has always been one of vital interest to those associated with the disposal of the city sewage at Schenectady, N. Y. The treatment plant was turned over to the City and operation commenced in January, 1915. The odors arising from the untuned Imhoff tanks, trickling filters, and sludge beds caused several property owners in the vicinity of the works to bring suit against the City to restrain it from operating the plant. The complaint stated an action for an injunction with fee damages, in view of the "exceedingly disagreeable, obnoxious and unhealthy odors" wafted to the plaintiffs' lands, about 1 500 ft. from the plant site. The action was begun in July and tried in November, 1915. Many lurid descriptions of the horrible odors in question were presented by witnesses. One day during the trial the Court and attorneys visited the plant. The cold weather had caused a cessation of intense bacterial activities in the treatment units and no odors of any kind were detected at a distance of less than 100 ft. It was the opinion of the Court that it had not been shown that the City could not run the plant without nuisance in the future and an injunction was denied. Fee damages were not allowed as no injury to health or no expense had resulted from the period of nuisance. The Court, however, advised the plaintiff that another action would be entertained if a repetition of such nuisance occurred.

Thus, at the beginning of 1916, a grave problem confronted the City. On the ability of the operators to control the odors from the plant depended the life of an installation that had cost approximately $500 000. The success that has resulted from the intelligent and careful manipulation of the treatment units is best shown by the fact that no formal complaint has been lodged against the plant in the past nine years, although most of those who signed affidavits of avowed nuisance in 1915 are still residents of the district in which the plant is located. It will be interesting to note the conditions existing at the treatment works at the present time.

Crude Sewage.—The crude sewage that comes to the plant is representative of the average American weak domestic flow. It seldom contains more than

200 parts per million of suspended solids and the daily variation is slight. The sewage is fresh and always contains an appreciable quantity of dissolved oxygen. About 65% of the total flow treated is pumped at a station about 2 miles from the works and little, if any, unbroken feces reach the plant. The odor of the liquid is soapy and has no tendency to become disagreeable unless the flow in the trunk sewer is greatly diminished and the sewage takes on stale characteristics on its way through the conduit. Whenever the pumping station is not in full operation, the flow in the trunk sewer is maintained by cutting in a 24-in. line that ordinarily is by-passed to the river because of the inability of the plant to handle the entire sewage flow of the City.

Bar Rack.—The crude sewage is passed through a stationary coarse bar rack that is removing on the average ½ cu. yd. of wet trash per day. These screenings consist of rags, waste, paper, and all forms of garbage, which are disposed of on a dump. It has been found that this deposit gives off a distinct odor of garbage in the spring when it begins to dry out, but when dried it smells only like humus. In order to overcome this putrefactive tendency, the deposit is spread, covered with dry hypochlorite of lime, and then composted with sand. The odor is thus controlled with great certainty. Present plans call for spreading the trash in the field and periodically plowing the accumulation under.

Imhoff Tanks.—After passing the bar rack, the sewage flows through the influent piping and submerged shear gates into the sedimentation compartments of the Imhoff tanks. This method of influx now replaces the old open influent channels that exposed the sewage to the air and retarded the velocity of flow to such an extent as to produce a deposition of solids in the invert. Overcoming this possible source of odor has done much to improve the sanitary conditions about the works.

The surface of the tanks are kept clean at all times by skimming, and the solids thus removed are thrown into the gas vents and hosed. The water supply of Schenectady contains about 160 parts per million of hardness and produces great quantities of insoluble soap curd that float on the tank surfaces. This grease has a tendency to give off a rancid fat odor under summer heat and great care is taken to skim off this material and to hose the sedimentation compartments frequently. All concrete surfaces and channels are kept clean by hosing.

Fine screens of ¼-in. mesh placed at the effluent weirs of the tanks and auxiliary screens of ½-in. mesh placed in the effluent channels leading to the dosing tanks prevent much solid matter from passing over to the filter stone and there producing odors.

The old influent channels now act as effluent carriers, and, therefore, one set of such channels is always out of service and filled with stagnant sewage. Metal flash-boards are placed at the dead weirs to keep floating solids from being blown into this quiescent liquor and there putrefying. In summer, there is a tendency for this stagnant sewage to become stale, and, therefore, flow is reversed once each week to avoid any chance for septization and the consequent escape of sulfide gases.

As is generally known all Schenectady sludge rises into the gas vents to produce a scum blanket that is more than 6 ft. thick in the spring. This scum is driven down with a 1¼-in. stream of water during the entire sludge-drawing period. The first hosing of the year produces a sour odor that may be caused by the fermentation of the great quantities of mash that clog the gas vents, but subsequent hosings result in only a slight tarry odor about the tanks. A phenol-derivative disinfectant was used with some success to offset the slight odors produced, but the cost proved prohibitive. The gas vents have been raised by means of wooden chimneys, 1 ft. high, closed with hinged covers. This retards the foaming over of the scum into the sedimentation compartments and prevents the dissemination of scum odors into the air.

Dosing Tanks.—On passing into the dosing tanks, the tank effluent spills over three 30-ft. weirs and cascades down steps that break up the liquid into a thin film and spray. Contrary to expectation, no distinct odor, except that of fresh sewage, has ever been detected at this point. The agitation has a distinct advantage. Tests show that there has been an increase in dissolved oxygen saturation of more than 40%, produced by the action of the weirs and steps. It would appear that, given a fresh, weak sewage, the greater the agitation, the less the tendency for septization and the production of odors. Slime and grease foam coat the walls and appurtenances of the siphon chamber and are hosed as soon as the characteristic musty odor is detected.

Trickling Filters.—The oxidation effected by the dosing tank overcomes the tendency of the liquid distributed on the filter stone by the Taylor nozzles to produce odors, and only slight evidence of such odors can be found to the lee of the filter area. Tests have further shown that there is an increase in dissolved oxygen in the tank effluent affected by the filter distribution piping before the liquid leaves the orifices of the nozzles which increase often amounts to more than 1 part per million.

The black gelatinous growth on the stone surface gives off no odor except when the beds are cut out of service, but this has not been done in two years. Hypochlorite of lime was used successfully during the summer of 1924 to control the development of *Psychoda*. This chemical removed much grease from the nozzle spindles and some of the gelatinous growth on the stone surface. From experience at Schenectady, the use of bleach on troublesome filters is to be recommended.

Sludge Beds.—Great care has been taken to keep all unripe sludge from the drying beds. Small beds, designed to drain the sludge force line, are used for draining test quantities of sludge, as any tendency for the material to give off odors can easily be controlled on the few square feet of these areas.

Conclusion.—All sewage plants have slight odors that are noticeable at the site to the trained observer, and the Schenectady installation is no exception. Odors at the plant are of inconvenience only to those on duty there, but the same odors when carried off plant property are of interest to the public. The prevailing wind at Schenectady blows over the plant in the direction of habitation only 1 000 ft. distant. The topography does not deter the transportation of odors; little obstruction is set up by trees. Thus, the absence

of complaint is sufficient proof that the plant is functioning without the production of such odors as would be carried beyond city property.

Most odors reported from sewage plants have little to do with the olfactory senses. They originate in the brain and are a result of over-active imagination. It is indicative of the prejudiced opinion the public has of sewage plants that so many express sincere surprise at the beauty and freshness of the sewage plant grounds and of the clean and inodorous condition of the treatment features. If any particular installation is functioning properly, the speaker would recommend that an invitation be given all taxpayers to visit the works. In no other way is it possible to combat the disgust that most laymen have for things related to the disposal of the liquid wastes of the community.

C. E. KEEFER,* ASSOC. M. AM. SOC. C. E. (by letter).†—The sanitary engineer is beginning to realize more and more the importance of controlling the odors from sewage treatment works. It is necessary not only to reduce the contamination and the pollution of natural watercourses, but also to render the air which one breathes pure and wholesome. This awakened consciousness of the engineer·is brought about by at least two factors: First, the public is demanding a more hygienic and satisfactory environment; and, secondly, the engineer himself is continually striving to advance the science of sanitation. For a number of years he has had at his disposal any one of a number of types of sewage treatment that would produce effluents to meet different chemical and bacteriological standards. However, it has been only within the past few years that the problem of the control of odors from sewage work has been treated and discussed before engineering societies. In addition to this, little practical work has been done. As regards the control of odors, sewage works are being designed almost the same to-day as they were ten or fifteen years ago.

The reason for this condition is not difficult to understand. The inherent nature of sewage is such that it will produce an odor and, under many conditions, an unpleasant one. Furthermore, during the course of treatment, the sewage is exposed to the atmosphere, and an excellent opportunity is afforded for the escape of unpleasant gases. One reason odors of any kind are so difficult to control is that little is known about them. As yet, it has not been determined definitely whether they are present in the gaseous or the liquid phase. However, the former assumption seems to be the correct one. Moreover, there is no satisfactory way to measure the degree of an odor. Not infrequently such small quantities of a substance have to be present to produce an odor that it is difficult to make any scientific study of the subject. Until some of these difficulties are overcome—especially determining the phase of the odor and establishing some standard by which odors can be measured—the engineer or scientist will be handicapped in their control and elimination.

If the available data are limited, however, efforts can still be made to reach a solution—even if it is a partial one. In the realm of sewage treatment much can be done to relieve the situation, as Mr. Skinner has ably pointed out, although such relief will not necessarily be complete. It is often possible to locate the sewage works in a sparsely inhabited district so that few people will be affected, to select that type of treatment which will produce a minimum of odors, and to pay particular attention to the details of design so that there will be a reduction in aerial nuisances. However, with the greatest care paid to design and operation, a sewage works consisting of sedimentation and oxidation units of any appreciable size will produce odors that will vary with a number of factors.

Not infrequently persons living in the vicinity of sewage plants will seek redress through the Courts for damages. Such has been the experience of a large sewage works of which the writer has knowledge. Since this plant was first put in operation, approximately $100 000 has been paid for damages caused in part by obnoxious odors. Thus far there seems to be no satisfactory and economical solution for this problem. The actual area of the various treatment units covers several acres, and even if the odors coming from the sewage could be collected and treated successfully, the cost of the undertaking would be prohibitive.

Experiments at Baltimore, Md.—At the Baltimore plant a careful study made by those in close contact with the work has led to the belief that most of the odors come from the preliminary settling tanks. After these tanks have been in service for a short time, a heavy scum forms, which gives off a strong and penetrating odor. If this odor could be eliminated successfully, the situation would be relieved.

With this in mind a series of experiments was begun in the summer of 1924, which consisted in treating the sewage gases with chlorine. One of the preliminary settling tanks was used for this experiment. This tank, which treats the sewage after it passes through bar screens, is approximately 420 ft. long and 103 ft. wide, with a working depth of from 9 to 14 ft. It has a capacity of about 3 800 000 gal. with a detention period of approximately 3.5 hours. The tank is divided into two compartments. The sewage, usually in a stale condition, first enters the smaller compartment, which is 105 ft. long, 103 ft. wide, and from 9 to 11 ft. deep. This compartment was covered with 2-in. tongue-and-groove planking and was thus practically air-tight. At opposite ends of the tank, two small wooden buildings were erected, one of which contained a blower and chlorine control apparatus. There was direct connection from this building to the air space between the top of the sewage and the under side of the wooden covering of the tank. The sewage gases passed into the building, and thence through the blower. The chlorine was applied to the discharge side of the blower which was connected direct to one end of a wooden conduit, or flue, 100 ft. long and 2 ft. square. This flue entered the second building, where an observer could study the treated odors.

The quantity of chlorine used was varied from 5 to 15 lb. per 24 hours, and the volume of gas treated from 630 to 1 480 cu. ft. per min. A number

of test runs were made both with the gases from sewage and from the sewage sludge. The tank was operated for a number of weeks so as to produce a thoroughly septic condition and a more intense odor. In order to ascertain the difference between the untreated and the treated gases, the observer first went into the building in which the blower was stationed; after observing the odor, he went into the second building into which the treated gases were passing. Here, he was able to isolate himself from the odors coming from the various parts of the sewage works, and thus breathe only the treated gases, which were being blown from the interior to the exterior of the building.

Observations Made.—One of the most noticeable characteristics of the untreated gas was that it no longer had the characteristic smell of sewage. The odor was considerably less objectionable. Perhaps covering the tank and excluding the sunlight played a part in this new condition. Another interesting change was that little scum formed on the surface of the sewage. This latter condition has prevailed during the warmest summer months up to the present time.

Observations of the chlorinated gases were made by a number of persons, and various opinions were expressed as to the efficacy of the chlorine. Some held that it improved the odor little, if any; others thought that it practically eliminated any unpleasant odor; and still others took an intermediate ground and contended that the application of the chlorine was a material benefit, but that it had not fully corrected the condition. No definite conclusions were reached. Additional experiments will be conducted during the summer of 1925, when it is hoped that more definite results will be obtained.

One of the unfortunate things in connection with any experiment on odors is that there is no satisfactory standard or unit by which they can be measured. There will often be a wide diversity of opinion with reference to any particular odor. For this reason, any conclusions that are reached, will lack a certain definiteness and finality. However, the experiments conducted at Baltimore are felt to be in the right direction, and will probably increase interest in the elimination of odors from other sewage plants. If tests of this character could be conducted at other places, additional data would be available from which more definite conclusions could be formulated.

C. G. GILLESPIE,* M. AM. Soc. C. E. (by letter).†—It is significant of an important exaction in the higher standard which the public is setting up in regard to clean surroundings and clean air, that all the papers presented before the Sanitary Engineering Division at the meeting of January 22, 1925, relate to the odor problem associated with industry and the disposal of putrefying wastes. A relatively untrodden field of endeavor, not at all simple, and fraught with the danger of the imposition of huge expense ultimately falling on the whole public, is opened in the engineering control of odors. In admirable fashion Mr. Skinner presents his own observations and directs engineers to seek out the principles affecting the behavior of odors so that efforts at control may proceed intelligently.

* Director, Bureau of San. Eng., California State Board of Health, Berkeley, Calif.
† Received by the Secretary, January 26, 1925.

The principles of course are best ascertained where odors from particular sources exist unmixed with those from other sources. This calls for considerable isolation, a condition not common in or near metropolitan areas. Then, too, local temperatures, air movement, variations in the sewage, kind and condition of works, and exactions of the neighboring public, will color experiences throughout the country.

In the absence of any guide to isolation requirements for prospective sewage plants in California, studies were begun in 1923, to ascertain the odor zones around a number of existing and typical plants. This was done by procuring a map of the particular locality showing the homes, roadways, and landmarks, and establishing on the map a number of points surrounding the disposal plant or area, at which points odors in noticeable degree had been experienced by residents. Corroboration of experiences is nearly always possible along public roadways. The extent of odor travel found from these tests is given in Table 1 and a typical nuisance map showing a survey for odors, in Fig. 1.

MAP OF
SEWER FARM AND VICINITY
PORTERVILLE, CALIFORNIA
Shaded Area Shows Zone of Nuisance
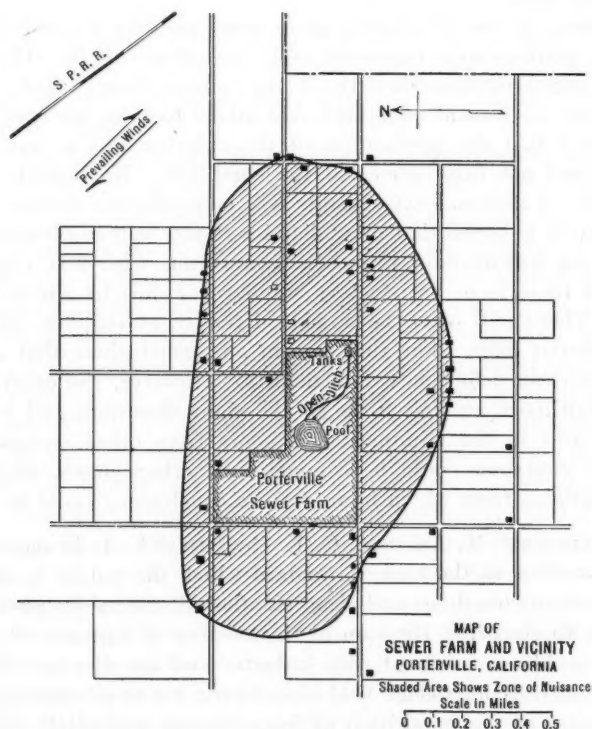Scale in Miles
0 0.1 0.2 0.3 0.4 0.5

FIG. 1.

In California, at least, the observations indicate that odors are most annoying in the summer and in the evening. Daytime inspections are invariably negative as to odors, except at most within a few hundred feet, whereas odors at evening travel many times as far. There is a noticeable tendency for odors to confine themselves to narrow, invisible ribbons of rather uniform

TABLE 1.—RESULTS OF ODOR SURVEYS AROUND CERTAIN SEWAGE TREATMENT PLANTS AND DISPOSAL AREAS IN CALIFORNIA.*

Place.	Population.	Treatment.	Disposal.	Nuisance Radius, in Miles.			Character of Sewage.
				Average.	Maximum.	Minimum.	
Hollister................	3 000	Raw sewage	Sewage farm	0.75	1.25	0.25	Contains gas-house waste and cannery waste
Vacaville (South Farm).	1 200	Raw sewage	Sewage farm	0.50	0.75	0.25	Domestic sewage
Vacaville (North Farm).	1 200	Septic tank	Sewage farm	0.75	1.00	0.25	Domestic sewage
Santa Rosa.............	13 000	Septic tanks	Sewage farm	1.00	1.25	0.25	Contains cannery, tannery and gas-house wastes
Fresno.................	55 000	Septic tanks	Sewage farm	2.00	2.50	1.50	Principally domestic sewage.
Selma.................	3 500	Imhoff tank	Sewage farm	1.75	3.00	1.00	Contains excessive cannery waste
Clovis.................	1 000	Imhoff tank	Sewage farm	0.37	0.50	0.50	Domestic sewage
Exeter.................	2 000	Imhoff tank	Sewage farm	0.62	0.75	0.50	Domestic sewage
Lemoore...............	1 500	Septic tank	Sewage farm	1.00	1.25	0.25	Domestic sewage
Porterville.............	4 500	Septic tank	Sewage farm (sandy)	0.75	1.00	0.50	Domestic sewage
Lindsay...............	2 800	Septic tank	Sewage farm	0.50	0.75	0.50	Domestic sewage
Corcoran..............	1 000	Imhoff tank	Sewage farm	0.50	0.75	0.25	Domestic sewage
Hanford...............	6 500	Imhoff tank	Sewage farm	1.00	1.50	1.25	Contains much cannery waste
Visalia................	6 500	Raw sewage	Sewage farm	1.50	1.75	0.75	Contains much gas-house and cannery waste
Reedley...............	3 500	Sprinkling filter	Kings River	0.37	0.75	0.25	Fresh domestic sewage
Kingsburg.............	1 200	Sprinkling filter	Kings River	0.75	1.25	0.50	Septic domestic sewage
Dinuba................	3 500	Septic tank	Sewage farm	1.25	1.50	1.00	Domestic sewage
Dinuba................	8 500	Sprinkling filter	Drain ditch	1.25	1.50	0.75	Domestic sewage
Paso Robles...........	2 000	Imhoff tank	Salinas River (bottoms)	0.37	0.50	0.25	Domestic sewage and hydrogen-sulfide swimming pool water
Tulare................	3 500	Septic tank	Sewage farm	1.5	2.00	1.00	Domestic sewage and much milk factory waste
Sonoma...............	1 200	Septic tank	Sewage farm	0.75	1.50	0.50	Domestic sewage

* By California State Board of Health, Bureau of Sanitary Engineering, C. G. Gillespie, Director, dated January 2, 1925.

intensity stretching out for as much as ½ mile, or more. These ribbons shift in direction and thus establish quite a remote periphery to the odor zone. High or strong winds, of course, dissipate odors rapidly. Low, gentle breezes, especially in the morning and evening, tend to shift the whole body of the nuisance zone only slightly leeward, as indicated by the relation of maximum to minimum nuisance radius. There is a tendency for odorous gases to seek low swales or pockets.

The quantity of organic matter, its state of decomposition, and the time or area exposed to the air before it disappears in the ground or is stabilized, markedly affect odor production. Cannery and creamery wastes, perhaps the most prominent industrial wastes in California, produce exceedingly strong odors. The waste from a modest plant is equivalent in its organic matter to the sewage of a large city. Odors are correspondingly intense in all the cases observed. Gas-house wastes impart a heavy but disagreeable odor in sewage, reminiscent of moth balls.

The sulfates in sewage appear to have a marked effect on the production of odor through their conversion to hydrogen sulfide. Along the outfall sewer in El Centro, Calif., intense sulfide odors emanate from the manholes. Sulfates in the 3-mile outfall drop gradually and uniformly from 330 parts per million ½ mile below El Centro to 160 parts per million at the Imhoff tank, and 155 parts per million in the tank effluent. It is curious to note that the sulfate-consuming organisms are reported to accumulate in this outfall until it becomes nearly choked. Application of copper sulfate at the head of the outfall destroyed and loosened the growth, allowed it to float down the outfall, and temporarily corrected the odor.

At Coalinga, Calif., where the sulfates in the water supply have approximated 1 000 parts per million, a crust of precipitated yellow sulfur has formed below the water line in the outfall sewer beyond the septic tank and the concrete of the tank itself is practically destroyed, probably due to the attack of associated acids. At the outlet of the tank intense odors of pure hydrogen sulfide are given off.

Long force mains between pumping plants and sewage treatment works of small towns in California contribute noticeably to the production of odor for the reason that over-size pumps must be used to prevent clogging and pipe lines are correspondingly large. The time of passage through this section of the sewer system may be as much as five times that through a gravity sewer of the same length, and the sewage not only ages as much as 3 or 4 hours in them, but becomes charged with putrefactive gases given off in the sewer.

For the same reasons, flat sewers, and sewers in which the manholes have a depressed bottom ostensibly for intercepting sediment, have intensified the production of odor. The writer's experiences with high groves of sheltering trees, reinforced by a 30-ft. levee, is that these barriers failed to confine the odors. One experience with the housing of sewage treatment works was not satisfactory. This was over a small Imhoff tank and sprinkling filter at Los Gatos, Calif. The housing so intensified the temperatures in the sprinkling

filter as to interfere seriously with its results. The housing was opened and the plant subsequently abandoned because of the odor annoyance to neighbors.

Experience at Dinuba, Calif., where a sprinkling filter and Imhoff tank superseded a septic tank and sewage farm, tends to show that the zone of odor was practically the same in both cases.

As to the activated sludge process the writer has made observations on only two plants, at Lodi and at Pasadena, Calif. The Lodi plant treats the sewage of a town of 7 500 people, including gas-house waste, and in summer also the waste from a 50-ton peach and tomato cannery. The sewers in Lodi are laid on extremely flat grades. Sludge is spread on sedimentary soil overlying hard-pan and when dry is plowed under as fertilizer. Odors are not noticed at a residence 500 ft. away at any time, nor have they even been noticed beyond a few feet from the plant or from the sludge area. Returning the sludge from the clarifiers to the sewer ahead of the fine screen, lessened the odors in the screen room, but caused more watery screenings.

The Pasadena plant treats a strong domestic sewage for a population of about 100 000. The worst fault of this plant is that it is undersized for a rapidly increasing load. Odors, however, were not a factor beyond 1 000 ft. or more, until deep lagooning of sludge was tried.

There seems to be a singling out of sewage odors for complaint by the neighboring public, probably through the old fear that sewage gases are dangerous to health. It seems to the writer that the public needs to know that sewage odors are merely a nuisance, more or less annoying and discomforting, not essentially different from the odor from corrals, pig pens, factories, fertilized fields, and the like. In equity, although one may be more intense or more widely annoying than the others, they are all nuisances differing only in degree. Unless there is considerable tolerance and this noticeable movement for cleaner air proceeds prudently, a financial burden altogether too heavy may easily result. In the final analysis the expense shifts to the whole public. Agitators may easily become inimical to the best public interest. A real odor nuisance generally depresses property values. If the total cost of abating all the nuisances responsible for that particular damage even approximates the estimated damage, it seems only logical that correction of the odor is in order and this principle may afford some means of judging the merit of a proposition to control odors.

T. CHALKLEY HATTON,* M. AM. Soc. C. E.—The speaker would like to challenge one statement† of Mr. Skinner, in which he says:

"Lagooning undigested sludge is likely to produce odors, for either scum will form or there will be insufficient water to cover the sludge, which will putrefy and give off odors.

"In general, lagooning should only be adopted in winter or in remote locations where no nuisance will be occasioned to neighbors."

At Houston, Tex., a nuisance has not resulted from the lagooning of activated sludge either in winter or in summer and little or no scum has formed on the surface of the liquor after some years of operation.

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* Chf. Engr., Sewerage Comm., Milwaukee, Wis.

† *Proceedings*, Am. Soc. C. E., May, 1925, Papers and Discussions, p. 850.

At several sewage treatment plants in England, visited this past summer (1924), undigested sludge was lagooned without creating a material nuisance which might be considered detrimental to the surrounding neighborhood. The large and numerous digestion tanks at Saltly in the Manchester plant, which contain highly concentrated undigested sludge, could not in July be termed offensive judged from 200 ft. away especially after the formation of the heavy top scum.

It is doubtless true that while the undigested sludge is being transported from one tank to another, the sulfuretted hydrogen escapes and creates an objectionable odor but when the sludge becomes quiescent the odors arising from its decomposition, when in the presence of water, are scarcely perceptible a few hundred yards away at most.

THE DETECTION AND ELIMINATION OF ODORS FROM OIL REFINERIES

Discussion*

By Messrs. N. E. Loomis and Edward Wright.

N. E. Loomis,† Esq. (by letter).‡—In carrying out most of the suggestions made by Mr. Weston, the writer feels certain that sanitary engineers will meet with hearty co-operation from the oil industry. In fact, in the largest and most up-to-date refineries they have already been carried out. This is for the reason that the escape of odors is immediately connected with other losses which the efficient refinery wishes to avoid.

For example, Mr. Weston has emphasized the importance of collecting the gases from the various distillation processes and burning them under stills, in order to obviate odor troubles and at the same time to effect fuel economies. As an alternative the scrubbing of gases with a solvent oil was mentioned with the objection that the gasoline recovered was so "wild" as to escape subsequently and thereby create a nuisance. As a matter of fact, in most of the first-class refineries all gas is collected from the distillation operations, scrubbed to recover the stable gasoline that it always contains, and then the dry gas is burned under boilers or stills. There are numerous methods for insuring that the recovered gasoline be free from dissolved gases which later might cause odor trouble. It frequently happens that the recovery of gasoline alone in such a manner pays a substantial profit in addition to having the fuel value of the dry gas, which value may amount to more than 20% of the total fuel consumed in the entire refinery. The combined operation of recovering the gasoline from the gas and using the dry gas as fuel represents an economy over wasting the gas that a large refinery cannot afford to neglect.

Mr. Weston recommends the use of a vacuum system on tankage. There are two kinds of tankage used in refinery operations—storage tanks for crude and refined products (which may be as large as 80 000 bbl. in capacity), and working tankage. The working tankage is smaller in size and, as a rule, comprises a much smaller number of tanks than the storage tankage. It is often feasible to tie the working tankage into a vacuum system as recommended, but the present construction of most storage tankage would make this imprac-

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ticable. Further, the escape of gases from storage tanks can be minimized in other ways so as to make it unnecessary.

Odors from the burning of oil under boilers and stills should not be objectionable provided complete combustion is obtained; here again there is a community of interest between the general public and the oil refiners. In many refineries it is the custom to keep a crew of men constantly studying the combustion conditions of the various pieces of equipment. Appearance of smoke at the stacks is an indication of improper operation.

Mention has also been made of the odors resulting from treating oils with acid and from the concentration of the recovered weak acid. In treating, the use of tight agitators and the mechanical mixing of oil and acid prevents the escape not only of odors but also of valuable fractions of the oils. The odor difficulties encountered in acid concentration are being minimized both by the use of proper types of concentrators and also by the proper conditioning of the weak acid to eliminate organic material.

In trying to eliminate odors from oil refineries sanitary engineers are fortunately facing a different situation from that in some other branches of industry. With the refineries they are dealing not with the prevention of a nuisance which can be corrected only at a monetary loss, but rather with a condition in which the economics effected will actually pay a profit in almost every case cited. This higher operating efficiency is certainly the strongest argument which can be used by sanitary engineers and public health officials in forwarding the work in which they are primarily interested. By proper co-operation based on the community of interest, sanitary engineers and refinery managers should be able easily to correct whatever troubles actually exist. In no case, however, do American conditions resemble in the faintest manner the picture that has been drawn of Russian conditions. The efficiency which has always maintained the American refining industry in a position of unchallenged world leadership would make such conditions in the United States absolutely impossible.

EDWARD WRIGHT,* M. AM. SOC. C. E.—The importance of the detection and elimination of odors from oil refineries is perhaps more fully realized by a comparatively small number of sanitary engineers in Eastern Massachusetts, in the vicinity of Providence, R. I., and certain districts in New York State and New Jersey, than by the profession in general. The number of complaints on account of odors from the refining of high sulfur oils since the World War has been unparalleled in the recent records of the Massachusetts Department of Public Health.

To indicate the size of the oil refining industry, it may be said that one of the refineries in Eastern Massachusetts has been the heaviest shipper on the New York, New Haven and Hartford Railroad System. Although the number of persons engaged in the industry is not great, an immense amount of capital is invested in the 500 to 600 refineries in the United States, and the industry is an increasingly essential one.

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Soon after the World War, the New England refineries used great quantities of Mexican crude oil which contained 1.5 to 6% sulfur. More recently, and simultaneously with greater knowledge and improvement of odor control apparatus, considerable quantities of crude oil rarely exceeding 0.5 of 1% of sulfur have been shipped from California and Texas and from other fields. Extensive prospecting is now under way in Venezuela where crude oil having a sulfur content from 1 to $3\frac{1}{2}$% is being obtained in increasing quantities. If large quantities of this crude oil are used, the objectionable conditions which arose with the use of Mexican crude may occur again unless adequate control devices are operated. Not all the sulfur is removed during the refining process; in fact, fuel oil may contain 3 to 4% of sulfur, and the speaker has noted hydrogen sulfide odors from the combustion of Mexican fuel oil which were more marked than the odors from a poorly managed sewage disposal works situated adjacent to the power plant. This indicates that the time may come when it will be necessary to remove some of the sulfur, possibly by chemical means, from certain fuel oils.

It may be of interest to describe briefly some of the methods and difficulties of operating odor-control devices that have come to the speaker's attention in examinations for the Massachusetts Department of Public Health. The installation of odor-control devices, like that of sewage disposal works, is only the beginning. Painstaking operation of the devices is essential. The operating force under which these various gas systems are controlled in one of the larger and more complete refineries in Massachusetts consists of ten men; a supervisor and a night supervisor, an experienced man and two helpers employed in the inspecting, cleaning, and changing of screens in the various gas lines, an experienced man and a helper employed in the inspection and maintenance of the water seals and recording instruments, and three gas-system operators working in three shifts, who make hourly inspections of the gas-collecting systems and the plant in general. Detailed reports of the inspections are submitted in writing or in tabular form to the supervisor at the end of each 8-hour shift, and the essential data are subsequently submitted to the management.

In the complete refinery, there are eight to ten recording instruments on the pipe systems for collecting non-condensable still-gas together with a number of additional instruments for determining temperatures and other data that are of value to the refinery, but of no particular significance in the odor elimination problem. Parts of the instruments must be renewed from time to time because of the corrosive effect of the gases. The records not only show the quantity of gas, but also immediate evidence of any defect in the gas collection or gas-pressure system, such as leaks or the formation of deposits. The instruments are placed so as to be visible, particularly to the operators of the gas exhausters.

In a refinery handling 15 000 to 20 000 bbl. of crude oil daily the non-condensable still-gas may amount to 1 500 000 cu. ft. per day, but the quantity varies greatly from hour to hour even when the stills are operated continuously.

The quantity of air in the non-condensable still-gas is estimated three times a week at one of the refineries in Massachusetts, and determinations are made less frequently for hydrogen sulfide and other constituents. The quantity of air varies from 10 to 35% by volume in the low-pressure still-gas, and when a weekly run of about one-third Mexican and two-thirds Californian crude is being used, the hydrogen sulfide varies from 3 to 15% by volume.

The gas pressure lines leading to the boilers are provided with one or more water seals which must be kept under constant supervision, with a seal of 2 in. of water. Running water is usually provided in these seals to compensate for evaporation, and care must be exercised to prevent freezing.

In disposing of the gases of evaporation from the crude oil storage tanks an equilibrium in pressure is provided on the tanks while they are being filled, whereas on the storage tanks for hot oil a constant vacuum of $\frac{1}{4}$ in. to 3 in. of water is maintained. U-tubes are provided at various points throughout this system, and in the winter these tubes are filled with a solution of gas-oil to prevent freezing. The liquid in the U-tubes is usually colored with red ink to facilitate reading, and readings are made hourly and properly recorded.

Thirty-mesh Monel metal screens are installed on the gas-line outlets of all the oil tanks and containers mainly for fire protection. The screens are given several inspections weekly, which indicate that the surfaces sometimes quickly become coated with deposits, largely of sulfur. The life of these screens averages not more than three weeks and proper management requires that they be kept in proper condition in order to prolong the periods between cleaning and renewal of the gas lines. At one of the refineries in Massachusetts, 3-in. gas lines on the hot-oil, storage-tank systems have recently been replaced with 6 and 8-in. lines because of deposits. The gases of evaporation are passed through suitable seals in which running water is maintained. The water seals on top of the tanks require inspection twice daily.

The quantity of air in the gases of evaporation, which is estimated three times weekly, varies from 40 to 80 per cent. Additional determinations are made less frequently for hydrogen sulfide and other constituents.

The maintenance of the odor-control devices in the plant for chemically treating raw gasoline is also troublesome, particularly because of the acids contained in the gases. All parts of this apparatus are inspected frequently for leaks.

Odors often result from deterioration of the metal parts of refining apparatus allowing the escape of the contents. Corrosion and deposits made up largely of sulfur present great difficulties in operating the gas-disposal systems at oil refineries, particularly in the non-condensable gas lines from the low-pressure stills where the gas contains as high as 35% of air. Corrosion is less marked in the high-pressure still lines where air is not present. The mixture of the air and gas with sulfur compounds readily corrodes wrought-iron pipe so that its average life is not more than three to four years. The mixture of the gases of evaporation from various grades of petroleum products and air also corrodes the tops of the metal tanks so that it is necessary to replace them every three to five years. In gas-holders for storing the non-

condensable gas, the corrosion is also marked and careful inspections and renewals of parts of the holders are necessary in order to prevent leaks.

Regular inspections are made of the still settings to determine the presence of breaks in the walls due to the contraction and expansion of the stills, and when the stills are used on the batch process, as is necessary when the heavy crude oils are refined down to coke, very careful attention is necessary to prevent leaks and to provide good combustion of the fuel beneath the stills.

The management of one of the refineries in Massachusetts requests the salesmen and others to make such observations in the locality about the refinery as are reasonable and to call up the management if offensive odors are noted. A representative of the Odor Control Department is on hand at all times to make observations in the field concerning complaints and to determine and correct the cause.

FINAL REPORT OF THE SPECIAL COMMITTEE ON STRESSES IN STRUCTURAL STEEL

Discussion*

By Messrs. Edward Godfrey, Jasper O. Draffin, E. G. Walker, and W. E. Belcher.

Edward Godfrey,† M. Am. Soc. C. E. (by letter).‡—The report of the Special Committee on Stresses in Structural Steel contains recommendations that should not go unchallenged. It is a serious if not a revolutionary matter to increase unit stresses 25 per cent. Furthermore it is well nigh impossible, once a practice has been established which breeds wrecks by the score, to induce the profession to abandon that practice for safe methods of design. Witness the long established practice of designing dams in total disregard of under pressure and pressure in the horizontal joints and the long list of failures of dams because of the reality of this pressure; then witness the recent flouting of under pressure as an element in the design of dams by authoritative utterances.

To show justification for radical change in design there must be some strong reasons set up—something that will stand the light of analysis. Judgment is not a reason. Majority vote never made a truth nor repealed one.

Behavior in service is cited in the report as one of the reasons higher stresses can safely be adopted, and industrial buildings are referred to where high unit stresses were used in the design.

Behavior in service is competent evidence, provided that service is full service and not nominally performed. It is very rare that any building receives the load for which it was designed. For example, take an ordinary six-story building 100 ft. square, designed for 60 lb. per sq. ft. on the floors. It would require 60 to 100 carloads of material and a deep snow on the roof to load that building to capacity. Some years ago the writer criticized the design of a school building because the floors were good for only 20 or 30 lb. per sq. ft. of floor area. The builder calculated what a school teacher and fifty pupils and desks would weigh and by this argument convinced the Board that the floors were safe.

The argument that some engineers are willing to take a longer chance on an existing bridge and continue its service when unit stresses are 24 000 to 26 000 lb. per sq. in. is one of psychology rather than engineering. It is true, however, that bridges do sometimes carry the full loads for which they are designed and carry those loads frequently. Such cases are rare, however.

Even railroad bridges seldom carry the rolling loads shown in the specifications. Highway bridges almost never receive their full loads in the trusses.

In 1897, the writer condemned the Seventh Street Bridge in Pittsburgh, Pa., as being unsafe for use in carrying street cars. This bridge was a double-chain suspension bridge, the eye-bar chains being 10 ft. apart vertically and connected by systems of web members which prevented their acting independently in carrying suspension loads. The condemnation was based largely on the observed fact that the eye-bars in the upper chains were buckled at mid-spans, showing conclusively that they were not acting to carry load as a suspension chain and that the lower chain was carrying all the load.

This bridge was designed with the idea that the two chains would be of equal or nearly equal value, as they were both of the same section. The load for which the bridge was designed was very much lighter than modern traffic would require. The bridge was subsequently used for street cars and "gave service" for some years. Before its recent removal all eye-bars in the top chain at mid-spans were bowed out in a large curve. It has been pointed to as a good example of design because it gave this service for so many years. One could argue from this that a unit load of 30 000 lb. or more on these iron eye-bars would be a safe load.

Before the first failure of the Quebec Bridge, in 1907, authoritative opinion as a sustainer of bridge loads ran rampant. Units around 24 000 lb. were going to be very common where civil and engineering laws did not hamper the designer. That wreck, though its cause was a careening traveler and not high unit stress, gave a decided check to the "kiting" of unit stresses; and another large bridge, designed with the same disregard of established safety precaution, was altered in its floor system so that it could not carry its design load.

The real reason for the acquiescence of engineers in the urge of steel fabricators seems to be lack of eye-opening failures.

The Committee's report states:

"All service deficiencies of early practice can be traced to causes unrelated to basic working stress, chiefly unskillful design of the general structural system, crude details, and insufficiency of bracing."

Does this mean that design is so perfect now that all uncertainties as to the distribution of stress have been eliminated? Does it mean that the perfect conditions of the testing laboratory have been duplicated in built-up bridge members?

On May 7, 1900, a girder span in the North Side, Pittsburgh, broke down under the load of a locomotive that it had doubtless carried many times. The bridge was more than thirty years old—an unanswerable argument for its safety from the standpoint of service. It was a tension break in the bottom flange. The unit tension in the wrought-iron plates was about 14 000 lb. per sq. in., and it was a fresh break. Who will say to what extent this failure is due to lack of uniform distribution of stress on the wide flange plates used and to what extent the confidence of the Maintenance Department may be blamed?

The Committee is wise to base its argument on low values rather than on averages in considering the yield point, with a view of using that property

of structural steel as a basis for safe unit stress, but it seems to have lost sight entirely of the fact that the commercially determined yield point is by no means the true elastic limit of steel. With the high speed used in the testing machine the "drop of beam" gives a yield point that is several thousand pounds higher than the true elastic limit of the steel. The writer has made tests on steel that ordinarily shows a yield point of 40 000 lb. per sq. in., and found that with a very slow speed the yield point, which in this case is doubtless the true elastic limit, was 32 000 lb. per sq. in.

If the Committee's value of 30 000 lb. be reduced by a proportionate amount, it is 24 000 lb. Where then is the factor of safety in a bridge where 20 000 to 26 000 lb. is considered safe?

The writer is in accord with the Committee in its declaration that secondary stresses may be neglected in buildings. He would also neglect such stresses in bridge work. He is not in agreement with the inference that the bending and twisting of columns at unbraced points should be classed as secondary stresses. A recent contribution by the writer to the discussion of this subject* sets this forth more fully.

The report states: "It is believed that yield points determined in column tests are not of importance in estimating the service value of columns." This is about the most important pronouncement of the Committee. If the yield point in a column test (the point where the column takes a permanent set, and begins to fail) is of no importance in compression members, the most treacherous and uncertain of all structural members, engineers had better revise their thinking. They had better discard consideration of the yield point in tension members also and design for stresses that are just under the ultimate.

With steel columns, as with hooped columns of reinforced concrete, the members may take a permanent set (or begin to crack in the concrete); and after this the load may be very greatly increased before failure. The ultimate load may be twice as great as the yield point. This range between the yield point and the ultimate is an index of the toughness of the column, and it is valuable in contributing reserve strength, where a brittle column loaded to the yield point would fail and bring down the whole structure with it; but the yield point should be considered the ultimate point of usefuless of a column. There is no doubt that repeated application of the yield point load will eventually produce failure in the column as it does in tension members.

Among tests on compression members made by Mr. C. P. Buchanan,† three wrought-iron members showed permanent sets (or yield points) at 17 000, 17 200, and 20 150 lb. per sq in., respectively. Five steel members showed permanent sets at from 12 600 to 25 500 lb. per sq. in. (average 19 700). The average maximum load of these members was 31 200 lb. per sq. in. Four columns had a ratio of slenderness of 46 and less, and the fifth had a ratio of 97 (ultimate strength 27 790 lb. per sq. in.).

These were well designed bridge members, and the tests were made by one of the best and most experienced shop inspectors of his day, so that the shop

* *Proceedings*, Am. Soc. C. E., April, 1925, Papers and Discussions, p. 644.
† *Engineering News*, December 26, 1907.

work must have been good. There is certainly no justification for the Committee's recommendation that unit stresses can be increased 25 per cent.

Some tests* on well-made bridge members described by J. A. L. Waddell, M. Am. Soc. C. E., show great regularity. Three of these with a ratio of 81 showed an average elastic limit of 16 800 lb. per sq. in. This is 23% greater than the "safe load" recommended by the Committee. Would the Committee abolish a factor of safety?

By ridiculing the "factor of ignorance" the Committee seems to assume that every fact is known concerning the stress in the structure. This in itself is a dangerous attitude. Men who are worst deceived are those who deceive themselves, and the best way to be deceived is to cover a design with an intricate cloak of mathematics and theory, so that one loses all sense of the practical outstanding fact that a structure is actually unsafe.

Two things are always referred to when exact knowledge concerning stresses in steel structures is discussed. One is impact and the other is secondary stress and the great strides made in its mathematical determination.

Every set of practical tests of secondary stresses shows that the actual stresses generally bear no relation whatever to the calculated stresses and that the safest structures are those designed in a manner to produce the greatest secondary stresses; and the structures are designed without regard to secondary stresses. In spite of this engineers bring secondary stresses into specifications to harass the designer.

Formulas state just how much impact is added by live loads of certain linear dimensions, and the formulas are based on the length of the portion of a span loaded. No impact tests ever made came anywhere near confirming these formulas. Dynamic effect is produced by such causes as roughness of track, lack of balance of engine drivers, nearness or remoteness to the track of the member considered, and in no degree whatever is it dependent on the length of bridge loaded.

The tests† recently reported to the Society by the Special Committee on Impact in Highway Bridges show impact values up to 150 per cent. No formula adds impact stresses so great as this. Our vaunted knowledge of every factor entering into bridge design is not even a good guess. We had better keep our feet on the ground.

JASPER O. DRAFFIN,‡ Assoc. M. Am. Soc. C. E. (by letter).§—The report of this Committee has served a useful purpose if it does no more than direct attention to the more or less empirical selection of working stresses. Any one who has occasion to examine old steel structures must marvel at times that some of the structures do not give greater evidence of distress under the high stresses which exist. The ability of steel to resist high stresses successfully, when these are not repeated enough times to be classed as "repeated stresses", is considerable.

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* *Proceedings*, Am. Soc. C. E., March, 1925, Society Affairs, p. 126.

† *Transactions*, Am. Soc. C. E., Vol. LXIII (1909), p. 250.

‡ Asst. Prof. of Theoretical and Applied Mechanics, Univ. of Illinois, Urbana, Ill.

§ Received by the Secretary, May 2, 1925.

In spite of the demonstrated ability of steel to adjust itself to local concentrations of high stress, however, the increase in the value of working stresses should be gradual. The report of this Committee proposes the use of higher working stresses for practically all members while the general tendency of Societies and Committees in their recent reports concerning working stresses in columns in bridges is to specify lower working stresses. It is true that the members in bridges are not directly comparable with those in buildings, but the evident opinion which has resulted in these recommendations to lower the working stresses in compression members in bridges should be given some consideration.

The Majority Report recommends higher working stresses with qualifications regarding loads, stress analysis, and material. These qualifications aim toward a highly desirable goal and emphasize the necessity for and the trend toward more careful stress analysis and·design. However, in the past many failures have not been primarily due to high allowable stresses but have been due rather to the neglect of important stresses and to improper design or details. These deficiencies in design and details may be attributed to a number of causes, among which are errors in calculation, lack of knowledge on the part of the designer of the correct method of stress calculation and design, failure to appreciate the significance of secondary stresses or their effects, and the desire to save on material or workmanship. If these deficiencies exist to-day, they will exist, to a lesser degree perhaps, in the immediate future. The same men who have designed unsafe buildings will continue to design unsafe buildings, or at least those with a low margin of safety, but they will probably use the proposed higher working stresses without regard to the limitations imposed by the Committee on·their use. It seems unfortunate to suggest that a competent designer using carefully controlled materials should be hampered in the economical design of a structure by the incompetent individual, but this does not necessarily follow from the adoption of the Minority Report. The suggestion of a standard by a Society does not compel every one to follow it, since it merely expresses the opinion of the Society as to what it considers to be safe values. The skillful designer may at any time adjust the working stresses so that they accord with the conditions under which he is working.

The change in accepted, safe standards should in general be made as fast as, and no faster than, improvement takes place in methods of stress computation, in proper design and detailing, in knowledge of the action of structures, in methods of fabrication, and in uniformity of material. The Minority Report recognizes that some increase in working stresses is advisable but would make the change more slowly than the Majority Report. The Minority Report should prevail.

E. G. WALKER,* M. AM. Soc. C. E. (by letter).†—The writer has read the report of the Committee with considerable interest and agrees in principle with the greater part of it, although he considers that the evidence put for-

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* London, England.

† Received by the Secretary, May 4, 1925.

ward in the Majority Report* is hardly sufficient, in itself, to justify fully the proposals that are made to increase the unit working stress to 20 000 lb. per sq. in. Although unit stresses up to 8 or even 9 long tons per sq. in. (17 920 lb. and 20 160 lb.) are used considerably in Great Britain, the majority of British official regulations are based on the use of a steel having an ultimate strength of 28 to 32 tons per sq. in. and the fixing of a working stress at one-fourth the mean tensile strength, namely, $7\frac{1}{2}$ tons per sq. in. (16 800 lb. per sq. in.). This figure is specified by the London County Council and the principal Government and local authorities, and is very generally adopted. Thus, the standard working stress in official British practice is only 5% higher than the usual American figure of 16 000 lb. per sq. in. To this extent, then, the Majority Report is a little misleading.

The writer considers that the method used in Table 1† for specifying axial compressive stress in columns would be more directly comparable if the expression was put in the usual standard form. It would then become,

$$20\,000 - 80\,\frac{l}{r},$$ with a maximum stress of 16 000 lb. per sq. in., instead of the

expression, $16\,000 - 80\,E$, used in Table 1. The modified form enables more direct comparison to be instituted with the numerical values in common use at the present time.

The writer is in full agreement with the principle, expressed in the Majority Report, of separating entirely the subject of working stresses from that of specified loadings, although he feels that the practicability of this course when framing any code of rules is a difficult matter. The fixing of a suitable working stress should depend entirely on the physical and mechanical qualities of the material and the general make-up and functions of the various parts of the structure, and also on the nature of the stress. These items are all fairly definite and although much is still to be learned about them they should in themselves form a sufficient basis for settling on the working stresses to be adopted. On the other hand, loading is a much more definite matter and there are few cases with which designers of structural steel work have to deal in which loads can be determined in any but an artificial manner. In the case of buildings it is usual to find that definite intensities of loading are specified for various parts of the structure, and familiarity with these different loadings is apt to make one lose sight of the fact that they are almost purely artificial in character and, in general, are not directly connected with the actual loads which come on the structure.

The safety of the framework of a building depends on two broad factors: (1) The load that is put upon it; and (2), the strength of the framing to withstand this load. Although these two are unrelated factors in the sense that the load put on the structure is not intrinsically connected with the ability of the structure to support that load and *vice versa*, yet, in practice, there must be a certain amount of connection between the two, since the aim in structural design must be to produce most economically the structure that

* *Proceedings*, Am. Soc. C. E., March, 1925, Papers and Discussions, p. 394.
† *Loc. cit.*, p. 403.

will meet the requirements. It is common experience that different designers, following the rules laid down by different codes, will produce noticeable differences of dimensions in structures designed to fulfill the same purpose. These differences may be, and frequently are, due as much to differences in the assumptions made in regard to loading as they are to the working stresses used. If, for example, a code specified that, for a floor in a building that is to be used for any particular purpose, a load of 200 lb. per sq. ft. is to be assumed, and another code states that a load of 300 lb. per sq. ft. is to be used, it naturally follows that the dimensions of the members in the second case must be considerably greater. Whatever may be assumed for the purpose of calculation, the actual loading on the structure is definitely settled when the building is put into use. This will produce definite stresses which may be greater or less than the computed stresses determined from the codes.

There is not much doubt that with a properly arranged structure, together with a sufficiently high degree of workmanship and the use of a grade of steel which complies with the general accepted specifications, stresses of a higher intensity than those normally used could be adopted. The stress used in designing a steel building is a computed rather than an actual stress; that is, particular loadings are assumed for floors, roofs, wind pressures, etc., and, in the ordinary way, they are assumed to be continuous over the areas concerned; from them sections are designed in accordance with a simplified approximate theory based on the limit of stress laid down in the specification. By this process the endeavor is made to get a structure in which the members are all stressed to the economical safe limit when the building is put into use. In many cases this result is not achieved at all—or only approximately. If generous estimates of loading are made the actual stresses may fall considerably short of the computed ones, as will also occur when, for simplifying calculations, assumptions "on the safe side" are made. It follows in such instances, therefore, that a nominal working stress of, say, 18 000 lb. per sq. in., might result in a section as heavy as that computed for a maximum stress of 16 000 lb. per sq. in., if the basic assumptions, allowances for impact and vibration, etc., and distribution of loads between the members of the structure were assumed on a sufficiently lavish basis. Therefore, although the writer agrees that the subject of the actual maximum working stresses allowable should be considered independently of any other matter relating to design, and believes that their values should be settled entirely with reference to the properties of the material and its distribution, he finds some difficulty in understanding how it is possible to lay down values to be used for purposes of calculation unless, at the same time, consideration is given to all other factors on which the computed stresses and the safety of the structure depend.

It is fairly well agreed nowadays that material such as structural steel can be stressed with safety nearly to its elastic limit without jeopardizing its strength. If, therefore, the actual stresses in a structure may be calculated to a sufficient degree of accuracy there is no reason why high working stresses cannot be used. With the increase in knowledge of the behavior of materials

and structures, it has been possible to approximate by calculation more and more closely to the actual stresses which are developed in the various members of a complex structure. At the same time methods of manufacturing have improved in such a way that there is greater certainty of the load and stress distributions approximating more closely those assumed in calculations. Hence, it is possible to increase working stresses in the same material without actually reducing the margin of safety, but unless adequate care is taken that the methods of design and details of construction are of the standard required for higher unit stresses, it is unsafe to permit their general use. The writer holds the view that it is unwise to formulate any recommendation as to the use of higher unit stresses in structural designing unless, at the same time, the other features on which the adequacy of the structure depend also have been fully considered, and the condition under which higher stresses may be used have been fully laid down. He would suggest, therefore, that the work which has been undertaken by the Special Committee on Stresses in Structural Steel should be enlarged to cover an inquiry as to the loadings which may be permitted in different cases and the quality of the workmanship which is necessary in order to allow of higher stresses being used with safety.

W. E. BELCHER,[*] M. AM. SOC. C. E. (by letter).[†]—The Committee has rendered a valuable service to the Society and to the Engineering Profession, having made definite recommendations for unit working stresses, based on experience, judgment, and the present state of the art. The report is practically an expression of judgment by experienced engineers forming the Committee, of whom eight recommend an increase of working stresses of 25%, roughly speaking, and four recommend an increase of $12\frac{1}{2}$ per cent. In the absence of additional test data bearing on the report, or of specifications for improving minimum steel requirements, the writer is in agreement with the Minority Report.[‡]

As recently as 1920 the Special Committee on Steel Columns and Struts after a long series of full-sized tests, recommended a column formula which limited compression stresses to lower values than customary practice, with a working stress of 12 000 lb. per sq. in. for use in columns up to a slenderness ratio of $\dfrac{l}{r} = 80$.[§] However, structural engineers generally continued to use formulas approximating $16\,000 - 70\,\dfrac{l}{r}$, with maximum values of 13 000 to 14 000 lb. per sq. in.

The increase now recommended by the Majority Report[‖] of the Special Committee on Stresses in Structural Steel seems to the writer to be too great a departure from the conservative expression of the former Committee.

[*] Structural Engr., Dwight P. Robinson & Co., New York, N. Y.
[†] Received by the Secretary, June 20, 1925.
[‡] Proceedings, Am. Soc. C. E., March, 1925, Papers and Discussions, p. 404.
[§] Transactions, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1583.
[‖] Proceedings, Am. Soc. C. E., March, 1925, Papers and Discussions, p. 401.

Building Codes.—The fact that the movement to adopt a basic increase to a unit stress of 18 000 lb. is already well under way by the American Institute of Steel Construction and the fact that the minority, as well as the majority of the Committee accepts this value as safe practice, makes it desirable for engineers to endorse the general adoption of the Institute's Standard Specifications for Building Codes at the present time. Already stresses higher than 16 000 lb. per sq. in. are in general use under favorable conditions of stress determination. The fact that the majority of the Committee recommend 20 000 lb. will lead to its further use on special work, such as industrial plants, isolated buildings, export work, etc. Much higher stresses are customary, for instance, in transmission-line practice.

For a building code law, however, the writer believes that the increase to 18 000 lb., as a basis, should be the extent of the change at the present. As suggested by the minority of the Committee, comparatively few structures are designed by engineers, as compared to those for which builders and owners make their own interpretation of the code. Much thought has been given to finding a remedy for this condition. It should be accepted as axiomatic that structural designs should be supervised by structural engineers. The Engineering Profession has every right to insist on this as a matter of public safety, and, at the same time, the organization of building departments should be adequate for proper examination and approval. The adoption of a standard design code such as that of the American Institute of Steel Construction would tend toward uniformity of practice and greatly facilitate both design and approval.

Yield Point.—The use of the yield point as a basis in determining safe working stresses is an important feature of the report; it was to be expected from the 1924 Progress Report of the Committee.* Handbooks giving ultimate loads to which the user may apply his own "factor of safety" should be discontinued. Reported tests of articulated structures are convincing proof that stresses beyond the yield point in various members introduce rapidly mounting stress changes and failure.

It is likely that the yield point of standard steel will gradually be increased and that more accurate and standardized methods of determining the yield point will be developed, justifying a further increase in working stress. The possibility of securing such improvements may be a proper subject for the attention of the Committee. Special steels already permit such increase; for instance, on the Pacific Coast, medium open-hearth steel is available at no increase in cost, having a guaranteed yield point of 45 000 lb. per sq. in. and on which mill tests show from 47 000 to 48 000 lb. The use of this steel has been approved by the State Railroad Commission of California for transmission-line structures using a working stress under a maximum load condition of 30 000 lb. per sq. in. This is an excellent quality of open-hearth steel, passing "structural grade" specifications. Steels are also commercially available at this time from German rolling-mills having a minimum yield point of more than 40 000 lb.

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* *Proceedings*, Am. Soc. C. E., March, 1924, Society Affairs, p. 263.

Column Formulas.—The straight line column formula recommended by the Committee, $20\,000 - 80\,\dfrac{l}{r}$, with a maximum of 16 000 lb. per sq. in., fits so closely the curve formula of the American Institute of Steel Construction within a working range up to $\dfrac{l}{r} = 180$, that the Institute might well have adopted it for values below its maximum of 15 000 lb. The report of the American Institute of Steel Construction states that a straight line formula is "mathematically inconsistent". It might also be stated that the materials used and loading conditions are not mathematically exact and the variation between the curve and a straight line formula is small compared to the range between the compression formulas recommended by various authorities for the same class of work. The straight line formula is much easier to use and to check and is equally correct within working limits.

Combined Stress.—The Committee did not state definitely its allowable "over-run" in working stresses, due to combined loadings, including wind. The subject is discussed quite fully, but conclusions are not given. The value of 24 000 lb. is recommended by the American Institute of Steel Construction; this appears to be a reasonably high figure.

Rivet Values.—The rivet values given show refinements in bearing value which are not explained in the report. It would be of interest to know more about the tests that may have been consulted by the Committee in making its recommendations. From tests made on thin angles connected through one leg, the writer found a relationship between shearing and bearing stress values of about 1 to 2.5 instead of the ratio generally used of 1 to 2. The report recommends a ratio as low as 1 to 1.8 in the case of power-driven rivets in single shear.

By failure in bearing is included splitting through the end of the piece, breaking out through the edge, and similar detail failures, as well as failure by gradual elongation of the holes.

Good details being essential, the writer would consider the ratio of 1 to 2 safe in all cases, and very conservative in the case of double shear.

PROGRESS REPORT
OF THE SPECIAL COMMITTEE
TO CODIFY PRESENT PRACTICE ON THE
BEARING VALUE OF SOILS FOR FOUNDATIONS, ETC.

Discussion*

By S. J. Fortin, M. Am. Soc. C. E.

S. J. Fortin,[†] M. Am. Soc. C. E. (by letter.)[‡]—The tests made by Mr. Goldbeck agree with certain facts and observations described elsewhere by the writer.[§]

Mr. Goldbeck states:[||] "It is apparent * * * that, in general, the smaller the superimposed area, the greater is the unit of load required for equal penetrations", and, a little farther, "whether or not the relation will hold, it is nevertheless certain that whenever two different sized bearing areas are used, the larger bearing block will penetrate deeper than the smaller for the same intensity of load". His Fig. 2[¶] shows, for instance, that a load of 8.6 lb. per sq. in. on a bearing area of 4 sq. in. produces the same penetration as a load of 4.8 lb. per sq. in. on a bearing area of 12 sq. in.

What is the explanation of such a behavior? In his paper on "Researches on the Structural Design of Highways by the U. S. Bureau of Public Roads",[**] Mr. Goldbeck says:

"It will be noted that when small bearing areas are used, the intensity of pressure required to produce a penetration of 0.1 in. far exceeds that for large sized blocks. This is readily explained by the fact that when the large bearing areas are used a greater thickness of the soil is compressed, which contributes more toward the movement of the block than in the case of the small bearing area."

This explanation is incomplete. Should it not, also, be taken into consideration that pressures are not transmitted downward vertically, but that the area of pressure increases with the depth?

Assume two bearing areas (Fig. 6), one of 4 sq. in., as in Fig. 6 (a), the other of 16 sq. in., as in Fig. 6 (b), the load in Fig. 6 (a) being 400 lb. and that

in Fig. 6 (*b*), 1 600 lb., the intensity in both cases being equal to 100 lb. per sq. in.

The pressure due to the superimposed loads is not transmitted downward vertically through the various layers of soil, but rather follows an angle, *X*. Whether this angle is 20° or 30° or 40°, the writer does not know, but it must depend in a certain measure on the nature of the soil. If the unit load in both cases is 100 lb. per sq. in., the same unit pressure exists on the soil at Elevation *A*, the point of application of the loads, but at any given depth, as Elevation *B*, the unit pressures are different, that in Fig. 6 (*a*) being evidently less than that in Fig. 6 (*b*).
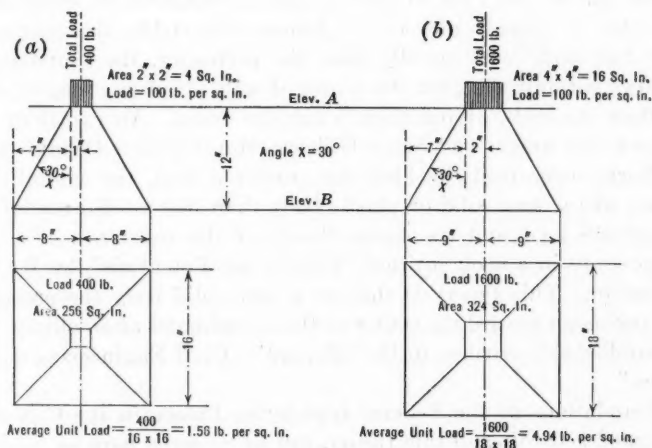

FIG. 6.

Translating this language into figures, using a depth of 12 in. and an angle of 30°, a sustaining area, at a depth of 12 in., of 16 by 16 in. = 256 sq. in. for Fig. 6 (*a*) and of 18 by 18 in. = 324 sq. in. for Fig. 6 (*b*), is found:

$$\text{Average unit pressure, Fig. 6 }(a) = \frac{400}{256} = 1.56 \text{ lb. per sq. in.}$$

$$\text{Average } \text{``} \qquad \text{``} \text{, Fig. 6 }(b) = \frac{1\,600}{324} = 4.94 \text{ ``} \quad \text{``} \quad \text{``} \quad \text{``}$$

Of course, this is an extreme case and, furthermore, the assumed angle of 30° may be too great. If, however, the assumption is correct it shows that the present method of designing foundations, where the loads differ widely in amount, is wrong. The unit of pressure for light loads should be higher and for heavy loads, lower. In other words, small foundation beds should be smaller and large foundation beds should be larger.

Another factor in the case is that at the depth, *B*, the pressure is evidently no longer uniform under the entire surface affected, but the unit pressure must be greater directly beneath the center of the superimposed load and lower near the edges.

The City of Mexico furnishes a practical demonstration of this statement. Part of the city is built on the bed of an old lake. The soil is evidently ejected

volcanic material, either deposited directly or washed down from the mountain sides surrounding the city. It has the appearance of clay, lies in alternate beds of various thicknesses, and is sometimes of brownish color, sometimes reddish. Occasionally, there is a thin bed of sand, boulders, and pockets which no doubt were originally filled with water. This substance is as soft as hard butter and weighs 70 lb. per cu. ft., that is, about 10 to 15% more than water. When it is taken out of the ground no water can be squeezed out by pressure, but if left exposed to the atmosphere it loses from 40 to 80% of its volume, becoming a gritty lump. This subsoil has an unknown depth; an Artesian well 800 ft. deep at the site of the National Theatre failed to reach bed-rock.

In that part of the City of Mexico under consideration there is hardly a building that is plumb and level. Almost invariably the center of large buildings has sunk more rapidly than the perimeter, the foundation taking a cup-shape; not only that, but the center of walls, if of any length, sinks more rapidly than the ends, giving them a concave form. An explanation of this phenomenon was sought and Señor Gilberto Montiel, then Deputy Minister of Public Works, conceived the idea that, under a load, the subsoil of Mexico acted more like a semi-solid or plastic body than like a solid material. After long researches he found the entire theory of the resistance of a semi-solid body to pressure in a book entitled "Théorie des Potentiels", by Boussinesq, a French author. This theory is that on a semi-solid body the pressure varies from the perimeter toward the center as the co-ordinates of an ellipse. Rankine has also studied this problem in his "Manual of Civil Engineering and Applied Mechanics."

The foundations of the Federal Legislative Palace in the City of Mexico were designed according to this theory, but as no walls have as yet been built it is impossible to test the accuracy of the assumptions. The sinking of buildings in the City of Mexico is not a settlement of a few inches, but of feet. A new structure at one corner has sunk at least 5 ft., and a very old wall has one end perhaps 10 ft. lower than the other. All the buildings do not sink more rapidly in the center than on the perimeter, nor do all the walls sink more rapidly in the center than on the ends, because local subsoil conditions may even reverse the process, but, generally, the sinking seems to follow the theory enunciated herein.

The concave deformation of a compressible body under a load can be illustrated graphically (see Fig. 7). It is assumed, in order to make the demonstration clearer, that the pressure due to a superimposed load follows an angle of 45°, but any other angle will give similar results. In this particular case, with four equal loads evenly spaced, the unit pressure at a certain depth near the center is four times the unit pressure at the end.

With good foundation material, not overloaded, this concave effect is not noticed because a material which can resist a certain pressure per unit of surface without apparent deformation, will resist a smaller pressure equally well. The reason that the concave effect in long walls is not noticed more often in constructions on weak subsoil, may be because walls well built act as a beam or perhaps as an arch, thereby creating an equilibrium between the resistance of the subsoil and the distribution of the intensity of the load on it.

No amount of reasoning or algebraic formulas will add much to present knowledge about foundations. Dependence must be placed on a close study of the physical properties of the materials and tests for the acquirement of new knowledge.
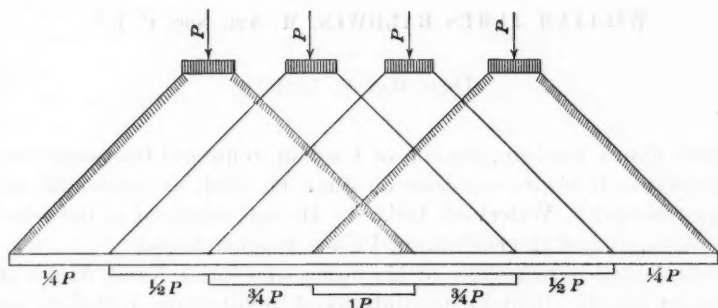

FIG. 7.

As to the variation of the unit pressures according to the co-ordinates of an ellipse, Fig. 5* shows that, for various loads on the same sized bed, the variation, or as Mr. Goldbeck calls it, the lateral distribution, is a curve having the shape of a bell—a curve remarkably similar to that representing the law of the probability of error.

Mr. Goldbeck is on the right path, and his researches and experiments will bring out some practical results. It will be interesting to have tests made on different grades of subsoil with a view of obtaining data on their behavior under various intensities of load, first, as to variation of unit pressure over the entire bed at various depths; and, second, as to the angle of transmission, X, or the curve of transmission.

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* *Proceedings,* Am. Soc. C. E., May, 1925, Papers and Discussions, p. 896.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

WILLIAM JAMES BALDWIN, M. Am. Soc. C. E.*

DIED MAY 9, 1924.

William James Baldwin, the son of Captain John and Giovanna Caterina (San Giovanni) Baldwin, was born on June 14, 1844, on shipboard, and his birth was recorded at Waterford, Ireland. He was educated in the schools of Boston, Mass., and of Charlottetown, Prince Edward Island.

In 1862, he entered the office of his father who was a Naval Architect and Surveyor for Lloyds (British), to study naval architecture, including navigation, drawing, engineering, and physics.

In order to acquire a practical knowledge of the details of ship construction, Mr. Baldwin, in 1864, accepted a position in the iron shipyard of Donald McKay, at East Boston, Mass., where he was engaged for two years on the construction of three monitors and the conversion of several blockade runners into cruisers. In 1866, he was made Assistant under Stephen Gates, Civil Engineer, in the construction and repair of iron ships at East Boston, but, in 1868, on account of the decline in iron shipbuilding, he turned his attention to general machinery. From 1870 to 1874, he served as Manager and General Superintendent of the Detroit Novelty Works, in charge of about 100 men, in the foundry, forge, and finishing shops.

From 1874 to his death, Mr. Baldwin engaged in professional practice as a Consulting and Designing Engineer, principally in connection with the construction and design of engineering plants for large public buildings, which included boilers, engines, elevators, pumps, small water-works, and ventilating and heating systems. Many of these buildings are of prominence in New York, N. Y., where he had established his headquarters, for example, the College of Physicians and Surgeons, the Vanderbilt Clinic, Sloane Maternity Hospital, and the old Tribune Building. He also installed engineering plants in many apartment houses; a department store; the Government buildings on David's Island in New York Harbor; the New York State Reformatory, Elmira, N. Y.; the Insane Asylum, Wilmington, Del.; Clayton Block, Denver Colo.; and the elementary schools at Honesdale, Pa. In the State of Michigan, he installed engineering plants in the State Prison at Jackson, the prison at Ionia, the State School at Coldwater, and the State Insane Asylum at Kalamazoo. Altogether, the number of boilers designed and set under his supervision exceeded five hundred.

He was also Consulting Engineer for the Department of Health, City of New York; Consulting Engineer and Designer for the United States War College at Washington, D. C.; for the United States Immigrant Station in

* Memoir compiled from information on file at the Headquarters of the Society.

New York Harbor; and for the United States Soldiers' Home in Tennessee; as well as the New York Telephone Company and the Empire City Subway Company.

From 1880 to 1889, Mr. Baldwin was Associate Editor of *Engineering Record* in its Department of Sanitary Engineering. He also served as Lecturer and Professor of Thermal Engineering at the Brooklyn Polytechnic Institute, Brooklyn, N. Y.

He was an Honorary Member of the American Society of Heating and Ventilating Engineers; a Member of the American Society of Mechanical Engineers and the Telephone Pioneers of America; and a Life Member of the Brooklyn Chapter of the American Institute of Architects. Mr. Baldwin was also a member of the Committee of the American Society of Mechanical Engineers that formulated standard pipe threads (known as the "Briggs formula"), for the United States and Canada in 1886, and of the Special Committee for Electric Screw-Thread Standards, as well as of the International Committee for the Formulation of an International Standard for Pipes and Fittings.

He was the author of a number of books, of which the following have been published: "Steam Heating for Buildings" (1881); "Hot Water Heating and Fitting" (1887); "Baldwin on Heating" (1890); "Data for Heating and Ventilation" (1897); "An Outline of Ventilation and Warming" (1899); and "The Ventilation of the School Room" (1901). He was also a contributor to the Dictionary of Architecture and Building.

Mr. Baldwin was elected a Member of the American Society of Civil Engineers on September 5, 1888.

GUION VICTOR BARRIL, M. Am. Soc. C. E.*

DIED APRIL 17, 1924.

Guion Victor Barril was born in New York, N. Y., on December 25, 1879. His paternal grandfather, John Joseph de Barril, came from Madrid when a young man, as Spanish Consul to Baltimore, Md., and remained in the United States, where he was well known in diplomatic and business circles. His maternal grandfather, William H. Guion, of the old "Guion Steamship Line," was one of the pioneers of the fast-going ocean steamers.

Mr. Barril was graduated from St. Francis Xavier College in New York, in 1898, and began his professional career as Rodman, with the City Surveyor of Baltimore, Md., remaining in this position approximately one year. From 1899 to 1901, he served as Rodman, Transitman, and Resident Engineer on preliminary and location surveys and on grade reduction for the Baltimore and Ohio Railroad Company.

In 1901, he held the position of Transitman and Chief of Party on grade reduction surveys for the Southern Railroad Company, and, in 1902, acted

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* Memoir prepared by E. S. Randolph, Assoc. M. Am. Soc. C. E.

as Chief of Party on location surveys for the Baltimore and Ohio Railroad Company. From 1902 to 1904, he was Assistant to the Chief Engineer on general construction work for the Northampton Portland Cement Company, and for a few months served as Topographical Draftsman with the Erie Railroad Company.

On July 19, 1904, Mr. Barril went to the Isthmus of Panama as a Civil Engineer with the Isthmian Canal Commission. He was first engaged on general engineering work and, later, was made Assistant in Charge of Street Improvement and other municipal work in the City of Panama.

Leaving the Canal Zone in 1909, Mr. Barril spent one year in private practice in New York and vicinity, and from 1910 to 1911, served as Assistant Superintendent on double-track construction for the New York, Ontario, and Western Railroad, for the MacDonald Construction Company of New York. He devoted the next three years to railway construction work in Brazil. Returning to the Panama Canal in 1914, he was associated with the Engineering Staff of the Fortification Division, and was in direct charge of the construction of some important works.

In November, 1921, Mr. Barril left the service of the Panama Canal to take charge of the Panama Construction Company, in the Republic of Panama. He served as Chief Engineer of that Company which was engaged on the construction of about 50 km. of highway for the Republic. While on this work Mr. Barril showed extreme tact and resourcefulness in dealing with a class of labor unfamiliar with the various phases involved in the construction of a first-class highway.

On the completion of the highway contract, Mr. Barril became associated with the United States Army on the Canal Zone, where he was engaged on miscellaneous construction projects until his death on April 17, 1924.

He was one of the pioneer builders of the Panama Canal, having arrived when the sanitary conditions were appalling and before modern science had dealt with the fevers and other diseases which are so prevalent in tropical countries. The improvements which have been wrought during the last twenty years, and have made the Canal Zone a winter resort visited by many tourists, are an enduring credit to, and an undeniable proof of, the efficacy of science, and much credit is due Mr. Barril for lending his assistance at a time when it was so greatly needed.

In August, 1900, Mr. Barril was married to Miss Virginia Alexander, of Winchester, Va. He is survived by his wife, a son, John J. Barril, and his mother, Mrs. Benton Gauldre-Boilleau, of Washington, D. C.

He was an able constructor, a diligent student, and very thorough in planning and keeping records of his construction work. He was prominent in his community, a member of the Panama Rotary Club, of the Engineers Club of the Canal Zone, of the Knights of Columbus, and of the Society of the Incas, the latter being composed only of pioneers in the construction of the Panama Canal.

Mr. Barril was elected a Member of the American Society of Civil Engineers on July 11, 1921.

THOMAS PETTUS BRANCH, M. Am. Soc. C. E.*

DIED MAY 28, 1923

Thomas Pettus Branch, the second son of Franklin Addison and Margaret Antonia Branch, was born at Tallahassee, Fla., on May 20, 1864. His father and grandfather were both Methodist ministers.

After completing his public school education he entered Vanderbilt University in September, 1881, and was graduated from that institution as a Civil Engineer in June, 1885.

In September, 1885, Mr. Branch assumed the duties of Professor of Mathematics and Engineering in the State Agricultural College at Corvallis, Ore., to which position he had been elected on the recommendation of Chancellor Garland of Vanderbilt University. At the end of his first year the ownership of the institution changed and the entire faculty resigned. Professor Branch served the Northern Pacific Railroad for the next three years, surveying and locating line and superintending the building of track through the mountains.

In 1889, he returned to Georgia and taught school for a few months in Meriwether County. Then, with a college friend, he surveyed and platted several towns in Tennessee, including Harriman. Between 1890 and 1894, he was in the service of the Savannah, Americus and Montgomery Railroad (now the Seaboard Air Line Railroad) and, as Field Engineer, built twenty miles of the line between Americus and Savannah, Ga. On the completion of this work, he entered the service of the Louisville and Nashville Railroad and was Resident Engineer at Guntersville, Ala., but his health having failed, he had to give up active field work.

In September, 1894, Professor Branch took charge of the Shellman, Ga., Public School and served in that position for one year.

In September, 1895, he became associated with the Georgia School of Technology as Adjunct Professor of Mathematics, and until his death was in charge of the Civil Engineering Department. For many years during this period he served as Secretary of the Faculty and Registrar. In 1920, the University of Georgia conferred on him the degree of Doctor of Science.

Dr. Branch had high ideals and lived a clean Christian life. He was an active member and for many years an officer in the Methodist Episcopal Church. Devotion to duty and loyalty to principle were the outstanding traits of his character. His long years of service in educational institutions developed in him a clear and balanced judgment of human nature, which, combined with a kindly and friendly personality, compelled the confidence and admiration of his associates and students.

At the Georgia School of Technology he shares the credit of having inaugurated the Co-Operative Educational Courses, and no member of its Faculty deserves more praise for its growth and influence. Dr. Branch's life was one of intensive and unselfish service. His influence on the undergraduates and

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* Memoir prepared by the late Paul H. Norcross, M. Am. Soc. C. E.

their subsequent careers during the twenty-eight years that he was connected with the Georgia School of Technology is reflected in the admiration with which his name is held by all who knew him. He lived and died beloved by all.

In 1895, he was married to Mary Susie Pharr who died in 1907. In 1909, he married Mina Pharr Thweatt who, with a son, Thomas Pharr Branch, and a daughter, survives him.

Dr. Branch was elected an Associate Member of the American Society of Civil Engineers on February 5, 1902, and a Member on May 5, 1915.

FRED AZDELL BRYAN, M. Am. Soc. C. E.*

DIED JULY 23, 1924.

Fred Azdell Bryan, a prominent citizen and one of the leading business men of South Bend, Ind., died suddenly, of septic poisoning, on July 23, 1924. Although he was but little past middle age, he had achieved a remarkable record and left a considerable sphere in the world better for his having lived in it.

Fred Azdell Bryan was born in Mechanicstown, Carroll County, Ohio, on December 21, 1867, the son of John and Anna Azdell Bryan, the latter of Columbiana County, Ohio. He attended the common schools and High School in Wellesville, Ohio. In 1886, he entered the Pennsylvania State College, from which he was graduated in 1890, as a Civil Engineer. As a student he was diligent, bright, and prompt, and his sunny disposition and staunchness of character endeared him to a circle much larger than his own class. He was never ostentatious, nor would he seek to gain favor or preferment through rivalry with his peers and associates. Friendship was to him above all considerations of personal advantage. Unselfishness was an outstanding trait of his beautiful character; and, for such a personality, a useful and successful career was to be foretold during his student days. His whole life was singularly free from those antagonisms and personal animosities which men of affairs so commonly encounter in conducting large business enterprises.

Soon after leaving college, in 1890, Mr. Bryan was engaged as an Assistant in an engineering party with the Pennsylvania Railroad Company. During the next two years he was employed as a Draftsman and Transitman with the Denver and Rio Grande Railroad Company. From 1893 to 1896 he was with the Edison General Electric Company, as Engineer and Superintendent of Construction in the building of approximately 75 miles of city and interurban electric railroads. During the next five years, 1896-1901, he served as Assistant Engineer with the Michigan Central Railroad Company and was stationed at Niles, Mich. In this connection his work had to do with extensive improvements and relocation of tracks for reduction of grades and curvature, bridge renewals, etc., carried out by the late Augustus Torrey, then Chief Engineer, of the Michigan Central Railroad.

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* Memoir prepared by W. M. Camp, M. Am. Soc. C. E.

About this time (1901) Mr. Charles A. Chapin, owner of the South Bend, Ind., Electric Company, undertook the promotion of an extensive scheme of hydro-electric power development on the St. Joseph River, to supply energy for electric lighting, electric railway operation, and general power purposes throughout the river valley. After searching for an engineer qualified by training and experience to take charge of this work, Mr. Chapin selected Mr. Bryan, and made him General Manager of the South Bend Electric Company, with headquarters at South Bend.

Mr. Bryan at once took charge of the design and construction of two dams and hydro-electric plants on the St. Joseph River, costing approximately $1 500 000, followed by the construction of a steam plant, in South Bend, costing $250 000, to take the peak of the load. In addition, he had charge of the operation of two hydraulic plants and another steam plant, of similar capacity, already constructed and supplying power to South Bend.

These extensive power developments for the operation of varied public utilities comprised hydro-electric plants at Twin Branch, Elkhart, Ind., Buchanan and Berrien Springs, Mich., with a power distribution extending from Michigan City, Ind., and St. Joseph and Benton Harbor, Mich., on the west, to Ligonier, Ind., on the east—a densely populated and manufacturing district approximating 2 000 sq. miles in area—and constituting one of the largest electrical utilities of the Middle West.

So faithfully and successfully did Mr. Bryan prosecute the purposes of this public utility enterprise that he remained with it as engineering and executive head for twenty-two years. In 1908 the South Bend Electric Company became the Indiana and Michigan Electric Company, with Mr. Bryan as its President. It was through his vision, initiative and executive ability that the original plant of the South Bend Electric Company was expanded to cover the additions previously mentioned.

In 1923 the controlling interest in the Indiana and Michigan Electric Company was sold to the American Gas and Electric Company. The new organization desired Mr. Bryan to retain the management of the property, but, largely owing to the condition of his health, he declined. Close application to the exacting requirements of multitudinous affairs had worn upon him, and he decided to seek relief in travel, with a view of engaging later in a business that would permit more time for recreation than he had previously been able to enjoy.

Early in 1923, shortly after severing his connection with the Indiana and Michigan Electric Company, Mr. and Mrs. Bryan started on an extended tour of the Orient. They visited Hawaii and Japan and traveled far into the interior of China. While in China, Mr. Bryan received a cablegram offering him an important position as a Consulting Engineer with the Studebaker interests, to serve in an advisory capacity with the Harris Trust and Savings Bank, of Chicago, Ill., which was financing the Studebaker enterprises, consisting of traction, gas, and water utilities in Illinois and Missouri. He accepted this offer, and on his return to the United States in the fall of 1923, became actively engaged in his new occupation, with an office

in South Bend, spending a part of his time, more or less regularly, in Chicago. He was thus engaged at the time of his death.

In his civic activities Mr. Bryan was hardly less distinguished than in his engineering and executive achievements. His admirable qualities of character, taken with his trained intellect and genius for organization, made him a splendid type of public-spirited citizen. Besides being a Director in the Citizens National Bank, of South Bend, he was a member of the Chamber of Commerce, the South Bend Country Club, the Knife and Fork Club, the Indiana and University Clubs, and of the Union League and the Indiana Society in Chicago. He also served as President of the South Bend Boy Scouts' Council, to which he gave much of his time and energy, a Director of the City Reserve Mission, and was a member of the First Presbyterian Church.

During the World War Mr. Bryan was placed in charge of the Third Liberty Loan for Northern Indiana, and the alacrity with which his district subscribed its full quota was credited to the executive ability and good will which he brought to bear upon this very pressing and indispensable financial project of the nation.

It is exceptional that the life of one engaged with as many business affairs as Mr. Bryan could be occupied so largely with the public enterprises of the community in which he lived. His masterful organization and management of the Boy Scout movement won for him not only the love and veneration of the boys but the admiration of the general public. In South Bend, with a population of about 80 000, the proper rearing of children has been studied as one of the most serious social problems of the community, and the zeal which Mr. Bryan put into this work was greatly appreciated and commended. His efforts in this service, in which he became so eminent, may have been prompted by recollections of the early years of his own life, or by the fact that he himself was childless. He had, by nature, a warm spot in his heart for boys. It is said, further, that he was constantly on the lookout for worthy boys ambitious for an education but lacking opportunities to acquire it; and that, in many instances, he found ways and means to place such young men in a position to realize their objective without letting them know their benefactor. It is deserving of all commendation that in his will he provided a substantial trust fund the income from which was designed to meet the cost of constantly maintaining four young men in college, the favored ones to be selected from among worthy boys of South Bend identified with the Boy Scout organization.

It would have been incompatible with Mr. Bryan's success as a public utility executive and leader in social service had he not been a man of independent opinions on public questions and policies. Although he was habitually frank and outspoken in the expression of his views, yet his manner of address was always so considerate and kindly that he seldom, if ever, offended. This can be said notwithstanding that there often fell to him the difficult task of conducting negotiations with State commissions, municipal councils, and other civic authorities regarding matters of franchise rights or rate adjust-

ments. He had a way of presenting his case that seldom failed to carry con-
viction. An open manner and conservative habit of statement, a conspicuous
sense of fair dealing, a direct method of getting to the point of a question
were characteristic of Mr. Bryan and accomplished for the interests that he
represented what most men of affairs would not attempt to carry through
without the employment of qualified legal talent. His willingness to consider
broadly the rights of others and a dignified manner of dealing were the criteria
of his business methods.

In his social life, as in his business career, Mr. Bryan always held to high
ideals and his tastes were cultural. He was married on June 9, 1896, to
Estelle M. McVicker, of Lisbon, Ohio, who survives him.

Mr. Bryan was elected a member of the American Society of Civil Engi-
neers on December 1, 1908.

JOHN FILLMORE HAYFORD, M. Am. Soc. C. E.*

DIED MARCH 10, 1925.

John Fillmore Hayford, the son of Hiram and Mildred Hayford, was born
at Rouses Point, N. Y., on May 19, 1868. His early education was secured,
while a farm boy, at the small stone schoolhouse not far from the farm.

Mr. Hayford was graduated from Cornell University in 1889, with the
degree of Civil Engineer, and immediately entered the service of the United
States Coast and Geodetic Survey at Washington, D. C., as a Computer. In
1892, he was made Assistant Astronomer in charge of field parties on the
International Boundary Commission, laying out the boundary between the
United States and Mexico, on which work he was engaged during 1892 and
1893.

On the completion of the Boundary Survey, he returned to the Coast and
Geodetic Survey as Aid, and, later, as Assistant in 1894 and 1895. He then
resigned to become Instructor in Civil Engineering at Cornell University,
which position he held until 1898, when he again entered the Coast and
Geodetic Survey as Expert Computer. In 1900, he was made Chief of the
Computing Division, succeeding Mr. Charles A. Schott, and, in addition, was
appointed Inspector of Geodetic Work. In this dual capacity he served the
Nation until 1909, when he resigned to become Director of the College of
Engineering at Northwestern University at Evanston, Ill., a position which
he held until his death. He died of a complication principally dropsical in
character, after an illness of three months.

While carrying on the regular work of the two departments of the Survey
over which he had supervision, Mr. Hayford found time to conduct many
original investigations. His theory of isostasy brought him international
reputation, for which he was awarded, in 1924, the Victoria Medal of the

Royal Geographic Society of Great Britain, an honor accorded to only two other Americans. His work with the transit micrometer and precision level revolutionized longitude determinations and precise level work, and he was a great factor in the adoption of the U. S. Standard Geodetic Datum, which has now become the North American Datum.

During the time that he was developing his theory of isostasy, Mr. Hayford made computations as to the figure of the earth, and his values of the lengths of the earth's semi-major and semi-minor axes, together with the ratio of polar flattening, were adopted in October, 1924, by the International Geodetic and Geophysical Union, as being the values which best fitted the various continents of the earth. He was twice selected to represent the United States as Geodetist at International Geodetic Conferences, once at Budapest, Hungary, and again at London, England.

As an active member of the Philosophical Society and of the Engineers' Club of Washington, he presented many papers. He was a member of the Cosmos Club of that city and took great delight in there meeting friends and visitors to the Capitol City.

At Northwestern University, as in Washington, Professor Hayford found time to do a prodigious amount of work other than that connected with his regular duties as Director of the College of Engineering. Especially notable was his work as Research Associate of the Carnegie Institution, studying the laws of evaporation, etc., underlying changes in lake levels on the Great Lakes, and in investigating factors affecting stream flow.

In 1912, Professor Hayford was selected by the Arbitrator, the late Chief Justice White, to head the Commission of Engineers to survey the disputed boundary between Panama and Costa Rica, and, later, was a member of the Commission of Engineers and Geologists sent to study the slides at the Panama Canal. In 1915, he was appointed by the President as a member of the National Advisory Committee for Aeronautics, and during the World War he was engaged with a group of scientists at the U. S. Bureau of Standards in devising instruments for recording airplane performance.

Professor Hayford was elected a member of the National Academy of Sciences, one of the highest honors granted to scientists in this country. He was also a member of the American Astronomical Society, American Association for the Advancement of Science, National Research Council of the American Geophysical Union, Western Society of Engineers, and the Society for the Promotion of Engineering Education. In 1918, George Washington University conferred on him the degree of Doctor of Science.

Although he was a scientist of the highest attainment, there was probably no field in which he took keener delight than in considering the problems of engineering education. He was honored by being elected President of the Society for the Promotion of Engineering Education for 1918-19.

As a citizen of the community in which he passed the last fifteen years of his life, Professor Hayford was most highly esteemed, and took an active interest in civic problems and affairs. He was a member of the Rotary Club,

University Club, Northwestern Chapter of Sigma Xi, and the Unitarian Church of Evanston, and of the Chaos Club of Chicago, Ill.

Professor Hayford was the author of "Geodetic Astronomy", a joint author of "The Adjustment of Observations by the Method of Least Squares," and author of numerous special reports, monograms, and articles dealing with geodetic, geophysical, geological, and educational subjects.

More than a quarter of a century of very intimate association with Professor Hayford, first as a member of the U. S. Coast and Geodetic Survey, and, later, as a member of the Faculty at Northwestern University, revealed the following characteristics to be uppermost in his life: Intense energy of body and mind, strict honesty of thought and purpose, and a hatred of all sham and pretense. He had the great faculty of making and holding friends; he seldom entered into anything without giving to it his fullest energies; and he obtained the keenest enjoyment out of life.

He was married on October 11, 1894, to Lucy Stone, of Charlotte, N. Y., who survives him, with four children, Walter S., Maxwell F., John B., and Phyllis Hayford. The latter is a Junior in the College of Engineering at Northwestern University, and the three sons have engineering degrees from the same institution.

Professor Hayford was elected an Associate Member of the American Society of Civil Engineers on May 6, 1896, and a Member on April 2, 1907.

OLAF HOFF, M. Am. Soc. C. E.*

Died December 24, 1924.

Olaf Hoff, the son of Martin C. and Gunhild Hoff, was born in Smaalenene, Norway, on April 2, 1859. He received his elementary education in the Real-gymnasium at Ringerike, Norway, and entered the Polytechnic Institute at Christiania (now Oslo), Norway, from which he was graduated in 1879 with the degree of Civil Engineer and with the highest honors ever granted by the Institute.

In 1879, Mr. Hoff came to the United States, and in the spring of 1880 entered the service of the Keystone Bridge Company as Assistant Foreman in one of the shops of the Company at Pittsburgh, Pa. In a few months he was transferred to the Drafting-Room, where he was gradually advanced, eventually becoming Assistant Chief Engineer.

From 1881 to 1883, he was in the employ of the Mexican Central Railroad, first as Bridge Engineer and, later, as Locating Engineer on the Tampico Division. From 1883 to 1885, Mr. Hoff served as Chief Engineer of the Shiffler Bridge Company, of Pittsburgh.

From 1885 to 1901, he was engaged in consulting and contracting practice in Minneapolis, Minn., acting, at the same time, as Western Representative of the Shiffler Bridge Company. In 1891, he built a steel railroad bridge across

the Mississippi River at Minneapolis for the Minneapolis and Western Railway Company, and devised the method of erecting this bridge without falsework. He designed a highway bridge across the Mississippi River at Muscatine, Iowa, 2 000 ft. long, with a 400-ft. cantilever span. He also acted as Consulting Engineer on several bridges crossing the Mississippi River at St. Paul, Minn., and made competitive designs for others, such as the Washington Bridge across the Harlem River, New York, N. Y., and two arch bridges across the Mississippi River, one at Nicolet Avenue and another at Lake Street, Minneapolis.

From 1901 to 1905, Mr. Hoff served as Engineer of Structures for the New York Central and Hudson River Railroad Company, in charge of bridges and buildings, and was identified with the renewal and reconstruction of more than four hundred bridges.

From 1905 to 1908, he was a member of the contracting firm of Butler Brothers-Hoff Company. This Company, as contractors, built the Michigan Central Railway Company's tunnel under the Detroit River between Detroit, Mich., and Windsor, Ont., Canada. The Railroad Company's plans called for a new type of subaqueous tunnel construction, consisting of twin steel tubes to be placed in a dredged trench and encased in concrete under water. Mr. Hoff gave all his exceptional inventive and sound engineering talent to the solution of a suitable design for the steel tubes and practical methods for floating and lowering them in place to correct alignment and grade, and for an adequate and economical encasement of these tubes with concrete placed in deep, swiftly flowing water.

From 1908, Mr. Hoff was engaged in consulting practice in New York, N. Y. The success of his past engineering performances had brought him an enviable reputation and he was therefore consulted by individuals and corporations regarding large problems of building construction, including subways, tunnels, harbors, and bridges.

The type of subaqueous tunnel used under the Detroit River and the methods employed in its construction had proved to be so satisfactory in every respect that Mr. Hoff secured, in competition with other tunnel types, the contract for the construction of the four-track tunnel carrying the Lexington Avenue Subway under the Harlem River in New York. This contract, awarded the McMullen-Hoff Company of which he was a member, was undertaken in 1912 and satisfactorily completed in 1915.

He acted as Chief Consultant to the Cunard Steamship Company, Limited, in connection with its proposed steamship terminal in the Port of New York and as Consulting Engineer to the New York Central Railroad Company on the design and construction of the A. H. Smith Memorial Bridge across the Hudson River at Castleton, N. Y., and of the Michigan Central Railroad Arch Bridge across the Niagara River, at Niagara Falls, N. Y.

Mr. Hoff's inventive mind concerned itself with many subjects. He invented methods for submarine pile-driving, reinforced concrete piles, grainbin construction of reinforced concrete, fireproof floor construction, etc. His

studious nature kept him fully abreast with the progress of his profession, and he possessed the inventiveness combined with a clear, analytical mind and the sense of fairness required for a successful engineer. These qualities made for him a great many friends and admirers in his profession, to whom he added many more among the laymen through his winning personality, his kind helpfulness, and his attention to other people's needs.

In 1885, Mr. Hoff was married to Josie Johnson Vea, of Stoughton, Wis. He is survived by Mrs. Hoff and five children, two daughters, Mrs. Olga Hoff Fernald and Mrs. Borghild Hoff Lyman, and three sons, Matthew, Olaf, and Trygve Hoff.

He was a member of the American Society for Testing Materials, American Association for the Advancement of Science, Engineers' Club of New York, National Geographic Society, Colony Club of New York, and Commonwealth Club of Montclair, N. J.

Mr. Hoff was elected a Member of the American Society of Civil Engineers on May 6, 1885.

MINARD LAFEVER HOLMAN, M. Am. Soc. C. E.*

DIED JANUARY 4, 1925.

Minard Lafever Holman, the son of John Henry and Mary Ann (Richards) Holman, was born in Mexico, Oxford County, Me., on June 15, 1852.

He attended Washington University in St. Louis, Mo., and was graduated from that institution in 1874 with the degree of Bachelor of Arts. The honorary degree of Master of Arts was conferred on him by his Alma Mater in 1905.

Mr. Holman served as Assistant Engineer in the Office of the Superintendent of Architecture of the United States Treasury Department for two years after his graduation; and then one year with the firm of Flad and Pfeifer, Engineers, at St. Louis. In 1877, he was appointed Assistant Engineer to the late Thomas J. Whitman, M. Am. Soc. C. E., Water Commissioner at St. Louis. He retained this position for ten years and was then appointed Water Commissioner by Mayor D. R. Francis, serving in this capacity for three terms of four years each. At that time, the Water Commissioner not only had full charge of the design, construction, and operation of the waterworks but was also a member of the Board of Public Improvements, which was charged with the design and construction of all public works for the City of St. Louis.

As Water Commissioner, Mr. Holman displayed rare ability in designing and constructing the extension of the water-works at the Chain of Rocks and the extension of the distribution system, involving an expenditure of about $10 000 000. This extension included a new intake located in the channel of the Mississippi River at the Chain of Rocks, about seven miles above the former intake at Bissell's Point; a tunnel under the river from the intake to

* Memoir prepared by Edward Flad, Baxter L. Brown and James C. Travilla, Members, Am. Soc. C. E.

the pumping station; a new low-service pumping plant having a capacity of about 100 000 000 gal. per day; six settling basins with a combined capacity of 180 000 000 gal.; and a brick and concrete conduit from the Chain of Rocks to Bissell's Point.

Mr. Holman's untiring efforts to reach the correct solution of the various problems encountered in this work, his honesty of purpose, his ready recognition of the meritorious work of his assistants, and his freedom from political influence in the organization of his Department, inspired those who served under him. As a member of the Board of Public Improvements, he upheld the traditions which enabled this Board to transact the public business with honesty and fairness. His broad knowledge of engineering helped to guide the Board in the proper solution of each new engineering problem. As Chairman of the Committee on City Lighting, he had charge of the conduit construction for the electric wires, and the preparation of the specifications and letting of contracts for the lighting of streets, alleys, and public buildings.

Mr. Holman kept well informed regarding engineering developments, especially those tending to improve the water-works. He installed high-duty triple-expansion pumping engines and down-draft boilers in the pumping plants, thereby materially reducing the cost of operation. Under his direction, electric cranes were installed in the engine houses and electric drives in the machine shop.

After the expiration of his third term as Water Commissioner, Mr. Holman accepted the position of Superintendent of the Missouri Edison Electric Company, which he held for five years. In 1904, he organized the firm of Holman and Laird, Consulting Hydraulic and Mechanical Engineers, and continued in active practice until ill health forced his retirement. He died at his home in St. Louis, on January 4, 1925, after a prolonged illness.

Mr. Holman was recognized as an authority in the field of water-works engineering, and was frequently consulted by various cities in connection with water-works problems. He served on the Board of Appraisers of Denver, Colo., which determined the value of the water system of that city and fixed the schedule of water rates.

He was actively interested in the American Society of Mechanical Engineers, having served as Vice-President in 1894-96 and in 1903-05, and as President in 1908-09. He was also an Honorary Member of the American Water Works Association and of the Engineers' Club of St. Louis, having served as President of the latter in 1888 and 1910.

On September 7, 1879, Mr. Holman was married to Margaret H. Holland, of St. Louis. His widow and three sons, Charles H., Minard H., and George R., survive him, one daughter, Mary Holman, having died in 1918.

The City of St. Louis is indebted to Mr. Holman for twenty-two years of faithful and unselfish service. He lived a noble and useful life; a seeker after truth, patient and untiring in his investigations, with a keen analytical mind, combined with unusual engineering ability, his acts inspired public confidence and general recognition of his honesty, ability, and fidelity.

Mr. Holman was elected a Member of the American Society of Civil Engineers on April 2, 1884, and served as Vice-President of the Society in 1905 and 1906.

JOSEPH MAYER, M. Am. Soc. C. E.*

DIED DECEMBER 13, 1924.

Joseph Mayer was born at Schriens, Vorarlberg, Austria, on February 11, 1855. His father, Anton Mayer, a native of Bludeur, Austria, was the owner of a flour mill and bakery and also of a hotel. His mother was Marie Bickel, also of Bludeur. Joseph Mayer was graduated from the Polytechnicum of Vienna, Austria, in 1880, with the degree of Professor of Mathematics and Descriptive Geometry for Realschulen.

After coming to the United States, Mr. Mayer was employed in the Chicago Office of the Delaware Bridge Company, serving for two years as Draftsman and for two years as an Assistant Engineer. Following a trip to Europe he was with the Union Bridge Company from September, 1885, to September, 1896, as Assistant Engineer in the New York Office from 1885 to 1890; in the Buffalo Office from 1890 to 1895; and as Chief Engineer in the New York Office in 1895 and 1896.

During this time he made designs, calculations, and estimates for innumerable bids, mainly for superstructures, and superintended detail plans for successful bids, among which were those for the following four large cantilever bridges: The Kentucky and Indiana Bridge across the Ohio River at New Albany, Ind. (Delaware Bridge Company); the Poughkeepsie Bridge across the Hudson River; the Kanawha River Bridge in West Virginia; and the Kentucky River Bridge.

From 1896 to 1908, Mr. Mayer practiced as a Consulting Engineer in New York, N. Y. Until 1901, he worked on a proposed railway suspension bridge across the Hudson River at 59th Street, making designs, plans, and estimates therefor for the Union Bridge Company. The late Alfred Noble, Past-President, Am. Soc. C. E., had agreed to be its Chief Engineer if the design met with his approval and if the project were successfully financed, but being a member of the Isthmian Canal Commission, he was unable then to give the study of the design his personal attention. Subsequently, he spent approximately two weeks in going over this work, personally re-checking all Mr. Mayer's assumptions and formulas and many of the stresses. He approved entirely of Mr. Mayer's design and plans, and so reported to the proposed financiers. The project failed to materialize, however, not because of difficulty in financing, but because of inability to secure the necessary legislation. The bill passed the State Legislature, but was vetoed by Governor Odell for the stated reason that it was a Special Bill disguised in the form of a General Bill.

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* Memoir prepared by Edwin Duryea, M. Am. Soc. C. E.

After the failure of the Hudson River Bridge project, in May, 1901, Mr. Mayer was engaged as Consulting Engineer for the Union Bridge Company, making plans and estimates for a proposed suspension bridge for Sydney Harbor, Sydney, Australia, both as to substructure and superstructure. This design won a prize. Later, under a second competition, Mr. Mayer (as Consulting Engineer for the Pennsylvania Steel Company) made another design and estimate of cost for a suspension bridge for Sydney Harbor, but neither of these bridges has been built.

Shortly thereafter (beginning in August, 1901), Mr. Mayer made a considerable study of the Brooklyn Bridge, New York, jointly with Edwin Duryea, M. Am. Soc. C. E. Following the alarming discovery by a man traveling across the bridge that seven short suspenders near the middle of the north main cable had failed, the District Attorney for New York County, the late Eugene Philbin, appointed Mr. Duryea to make an examination of the bridge and report on its safety and on the responsibility for its neglect. All traffic was stopped for a day or two while temporary repairs were being made. After a hasty inspection of the structure had been made, the District Attorney consented to the appointment of Mr. Mayer as an Associate Consulting Engineer. In the ensuing detailed and protracted study of all parts of the bridge, Mr. Duryea made the field examination and Mr. Mayer the stress analyses. The joint report was placed by the District Attorney before the Grand Jury, which ordered more careful and more frequent inspections of all parts of the bridge. This report was censured severely by the Bridge Department of New York City and its friends, and the Consulting Engineers spent much time for several months in refuting the attacks in various technical periodicals, many of which had published the report in full.

Subsequently, Mr. Mayer was Consulting Engineer on the construction of the power plant of the Ontario Power Company at Niagara Falls. He designed all the bridges and viaducts for the Buffalo and Susquehanna Railway Company and was engaged in other miscellaneous engineering work.

Leaving his consulting work in 1908, Mr. Mayer was Principal Assistant Engineer of the Quebec Bridge until its completion in November, 1917. He then retired from active practice and moved to Pasadena, Calif., where he made his home, occupying his time in the study of economics and social science in which he was much interested, until his sudden death.

Mr. Mayer was married in October, 1900, to Miss Catherine Proescher, of New York, who, with their son and only child, Joseph Edward Mayer, a Teaching Fellow in Chemistry in the University of California, Berkeley, Calif., and a brother in Austria, survives him.

Charles Macdonald, Past-President, Am. Soc. C. E., and for many years President of the Union Bridge Company, writes regarding Mr. Mayer as follows:

"I can say of his [Mr. Mayer's] standing as a designer of bridges that it was of the highest order of merit. I cannot call to mind the many plans he worked out for us, but I remember one particular design for which he deserves all the credit, namely, a suspension bridge to cross the Hudson River

at 59th Street, New York. It was accepted by the New York and New Jersey Commissioners, but the project failed to secure the necessary capital.

"The legislation of the Union Bridge Company in its later years made it necessary to dispense with Mr. Mayer's services, but I shall always remember with pride and gratitude his never failing attention to duty while I had the pleasure of being associated with him."

Ralph Modjeski, M. Am. Soc. C. E., one of the Engineers and Commissioners who had charge of the reconstruction of the Quebec Bridge, writes about Mr. Mayer's work there, as follows:

"He was employed as Principal Assistant Engineer in the Montreal office of the Board of Engineers and was particularly in charge of all calculations and designs. He was connected with that work from October, 1908, until its completion. His great usefulness came from the fact that he was a most unusual mathematician as well as an excellent designer."

Mr. Mayer was a frequent contributor to the *Transactions* of the Society, having been the author of six papers and having contributed many discussions which dealt not only with the subject of bridges, but also with other subjects, such as "Canals Between the Lakes and New York", "The Just Value of Monopolies and the Regulation of the Prices of Their Products", catenary trolley construction, centrifugal pumps and fans, principles of valuation, etc. He was nothing less than a genius in the practical application of higher mathematics to bridge design, particularly to the design of suspension bridges.

Mr. Mayer was elected a Member of the American Society of Civil Engineers on October 3, 1894.

CHARLES MILLER MORSE, M. Am. Soc. C. E.*

DIED JUNE 18, 1921.

Charles Miller Morse was born in Buffalo, N. Y., on January 11, 1854. He was the son of David R. Morse, a financier and merchant in that city, who for several years was President of the Erie County Savings Bank.

In 1870, Mr. Morse entered Yale University, intending to take the Classical Course, with a view to a literary career, but he developed a taste for applied science and studied Mechanical Engineering for two years. He supplemented his study at Sheffield Scientific School by practical shop training in the puddling mill of the Union Iron Works at Buffalo.

After leaving Yale, he was employed, in 1873, on a survey for the Buffalo and Jamestown Railroad, and from 1873 to 1874, was Transitman with the U. S. Engineer Corps on work along Lakes Erie and Ontario. In 1876, he went to Dunkirk, N. Y., where he learned the trade of machinist at the Brooks Locomotive Works, and in 1877 and 1878, he served in that capacity in the shops of the New York, Lake Erie, and Western Railroad Company at Buffalo, and also in the shops of the Lake Shore and Michigan Southern Railway Company at Cleveland, Ohio.

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* Memoir prepared by Charles Warren Hunt, Secretary Emeritus, Am. Soc. C. E.

Mr. Morse was afterward engaged as Draftsman and Assistant to the Superintendent of Motive Power with the Erie Railroad Company. In the latter position he had charge of the reconstruction of the locomotives of the Company from broad gauge to standard gauge, the work having been done at the Locomotive Shops of The Dickson Manufacturing Company at Scranton, Pa. From 1878 to 1881, he designed and superintended the construction of railroad shop equipments for the New York, Lake Erie, and Western Railroad Company at Buffalo and at Hornellsville, N. Y. From 1881 to 1882, he was Superintendent of the Crown Point Iron Company, at Crown Point, N. Y., and, in 1882, entered private practice in New York, N. Y. From 1883 to 1888, he also acted as New York representative for several manufacturers of machinery.

In 1888, he returned to Buffalo where he was engaged in private practice as a Mechanical Engineer and became head of the Buffalo Engineering Company. From 1890 to 1894, he acted as representative of Otis Brothers and Company, of New York, and, subsequently, as representative of the Babcock and Wilcox Boiler Company.

Mr. Morse was City Engineer of Buffalo for seven years. On December 26, 1901, he was appointed Deputy Engineer Commissioner in charge of the Bureau of Engineering of the Department of Public Works, serving until December 31, 1908, and receiving much commendation for his honest and efficient work. During his administration, the overflowed lands in South Buffalo were reclaimed, and an extensive report on the subject written by him in 1903, was published* by the Buffalo Society of Natural Science in 1908.

On August 6, 1907, the United States Government issued to him a license as Chief Engineer, and he was attached to the Naval Militia Steamer *Hawk* in the District of Buffalo.

In December, 1890, Mr. Morse was married to Kathleen Edgar, of Easton, Pa. Owing to the ill health of his wife, he bought a small fruit farm in Mount Dora, Fla., where he spent several winters, meanwhile retaining his business connections in Buffalo and New York. Mrs. Morse died on May 11, 1916. In 1917, he disposed of his home in Buffalo and moved to Florida permanently. At this time he gave up engineering practice and engaged in farming.

On April 3, 1918, Mr. Morse was married to Miss Mary A. Donohue, of Boston, Mass., who survives him.

He died at Mount Dora on June 18, 1921, and was buried at Forest Lawn Cemetery at Buffalo.

He was a member of the American Society of Mechanical Engineers, of the Sons of the American Revolution, the Army and Navy Club of America, Cosmos Club of Washington, D. C., Engineers Club of New York, Buffalo Club, and Columbia Yacht Club, and his genial personality and social characteristics endeared him to all with whom he came in contact.

Mr. Morse was elected a Member of the American Society of Civil Engineers on January 2, 1895.

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* *Bulletin*, Buffalo Soc. of Natural Science, Vol. VIII (1908), No. 2.

COLEMAN MERIWETHER, Affiliate, Am. Soc. C. E.*

Died October 8, 1924.

Coleman Meriwether, the son of Charles and Patti (Barbour) Meriwether, was born in Louisville, Ky., on July 14, 1875. He received his early education in private schools of that city and in the Lawrenceville Preparatory School, at Lawrenceville, N. J.

In September, 1894, Mr. Meriwether entered the Engineering Department of Snead and Company Iron Works, at Louisville, as Draftsman. In March, 1896, he resigned this position to become Manager of the Louisville Branch of the Pope Manufacturing Company of Hartford, Conn. In April, 1899, he entered the office of the Louisville and Nashville Railroad Company, at Louisville, as Assistant Engineer, where he remained until, in June, 1900, he became Draftsman in the office of the Chief Engineer of the Baltimore and Ohio Railroad, at Baltimore, Md.

In April, 1901, he was appointed Chief Draftsman in the office of the Chief Engineer of the Southern Railway Company (St. Louis-Louisville Lines), at Louisville, but in January, 1904, he was made Land and Industrial Agent and transferred to Birmingham, Ala. From January to September, 1904, he served as Chief Draftsman in the Chief Engineer's office of the St. Louis-San Francisco Railway Company, at Cape Girardeau, Mo., resigning to accept a similar position with the Seaboard Air Line Railway Company at Portsmouth, Va.

About this time, Mr. Meriwether became interested in protecting timber piles by using lock-joint pipe, and developed the sewer pipe joint known as the Meriwether system of lock-joint pipe. From April, 1905, until August, 1917, he served as President and Chief Engineer of the Lock Joint Pipe Company of New York, N. Y., and Ampere, N. J., and from August, 1917, to January, 1919, as Manager of the Cement Products Bureau of the Portland Cement Association, of Chicago, Ill. From January, 1919, to April, 1920, he was associated with Mr. H. R. Duckwall, of Indianapolis, Ind., in the development of high-pressure reinforced concrete water pipe to a satisfactory conclusion, and for almost a year thereafter he was interested in the promotion of this type of pipe at Louisville. In January, 1921, he organized the Meriwether Pressure Pipe Company, of which he was Vice-President and Engineer until his death.

In his efforts to popularize reinforced concrete pipe, Mr. Meriwether traveled throughout much of the civilized world and made many friends wherever he went. He was widely known as an inventor of concrete pipe joints, having obtained seven patents on reinforced concrete specialties, and as a manufacturer of reinforced concrete pipe.

On June 21, 1919, he was married to Miss Jessie Burks Scott, of Lynchburg, Va., who survives him.

He was a member of the American Society for Testing Materials, American Concrete Institute, Concrete Products Association, and the Engineering Insti-

* Memoir prepared from information on file at the Headquarters of the Society.

tute of Canada. He was also a member of the Engineers Club of New York and of the Country Club at Louisville, Ky.

In an obituary notice published elsewhere,* it was said that:

"A fitting epitaph for Coleman Meriwether may well include these thoughts: 'He achieved success in that he lived well, laughed often, was loved by many. He left the world better in that his work for civilization endures and has been an inspiration to others. His memory is revered by many friends'."

Mr. Meriwether was elected an Affiliate of the American Society of Civil Engineers on February 7, 1906.

WILLARD ADELBERT SMITH, Affiliate, Am. Soc. C. E.†

DIED NOVEMBER 29, 1923.

In the death of Willard Adelbert Smith, the nation has lost one of its best-known authorities on the history and status of railroad transportation in the United States. Although he had passed the traditional age of three score years and ten he had in no sense lost interest in the pressing questions of the day, his activity in portraying the truth of railroad conditions having been maintained to the time of his sudden failure in health, about a year before his death.

His acquaintance and influence in the railroad industry were far-reaching. He had met railway men in all grades of service and authority, from division operating officials to the highest executives, and was constantly being called into their councils. There were but few men, outside of those actually employed in railway service, to whom was given the opportunity to know as much of the railway industry from the inside as was the case with Mr. Smith throughout his long career.

Willard Adelbert Smith was born on a farm, near Kenosha, Wis., on September 20, 1849. He was the son of William Harrison and Mehitabel (Allen) Smith, who emigrated to Wisconsin from New Hampshire in 1841. His paternal ancestors came from England in 1640, and his mother's family came also from England at an early day and settled in the State of New York. His father was an Abolitionist whose home was a station on the "Underground Railway", serving as a hiding place for fugitive slaves.

Mr. Smith was educated in the common schools of Kenosha, the high school of Rockford, Ill., and Shurtleff College, at Upper Alton, Ill., from which he was graduated, in a literary course, in 1869. He next entered the Law Department of Washington University, at St. Louis, Mo., from which he was graduated in 1872. He was admitted to the Bar, in the State of Missouri, in 1870, and to practice in the United States Courts, in 1871. In later years, the degrees of A.M. and LL.D. were conferred on him by Shurtleff College.

From his early choice of occupation it was the destiny of Willard Smith to become a publisher; and his ability to master the varied scope of his activi-

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* *Concrete Products*, November, 1924, p. 72.
† Memoir prepared by W. M. Camp, M. Am. Soc. C. E.

ties, both private and public, throughout his life, developed from his experience with the publishing business as the nucleus. He first entered publication work in 1871, as Editor and Publisher of the *St. Louis Railway Register.* Three years later he acquired the ownership of the *Railway Review,* of Chicago, Ill., which he retained until he died. Thus, at the time of his death, his ownership and management of the *Railway Review* lacked but a few months of a full half century. In 1897 the name of the publication was changed to the *Railway and Engineering Review,* and in 1914 it was changed back again to the *Railway Review.*

In his charge and direction of the publication of this weekly railway newspaper and magazine Mr. Smith was, except during a few short periods, editorial writer as well as the head of the business management. Liberally educated and versatile, he early acquired a broad knowledge of the railroad transportation development of the country, in the technique of construction, equipment and operation, as well as of the financial and executive management of the properties. In such an experience he saw the railroad mileage of the nation expand from about 55 000 to 260 000; and the progress of Government regulation of the railroads he had followed, studied and written of, critically and ably, from its birth to the present involved and shifty situation.

Under the guidance of Willard Smith the attitude of the *Railway Review* on railroad questions was independent, espousing the cause of the railroads against the many unjust assaults of legislative authority, in the past, and freely criticizing railroad officials for errors of management or policy whenever that view seemed sufficiently well supported by the facts. In his editorial ideas and expressions the man was entirely frank and fearless. The motto of his editorial attitude on all questions was "to think clearly, to speak plainly, and to say the thing that ought to be said." His command of English was excellent, he had the habit of speaking plainly, and when his thought was written down it was so clearly and so courageously expressed that there was no need for "interpretation." Being an accomplished public speaker his voice, as well as his pen, was often called into service.

It was these abilities and qualifications that brought about Mr. Smith's appointment on July 27, 1891, as Chief of the Department of Transportation at the World's Columbian Exposition, held in Chicago in 1893. A broad and careful canvass of the views of railroad executives, by the Exposition authorities, as to the logical man for this work decided the matter for Willard A. Smith, and he undertook the work enthusiastically. For a whole year he traveled through Europe making arrangements for the foreign exhibits. How well he succeeded has become a matter of history. While his editorial ability had gained for him a national reputation, the organization which he developed to display the material progress of railroads throughout the world made him a man of international renown. Among the outstanding features of the result were the historical exhibit of equipment of the Baltimore and Ohio Railroad; the extensive display of track and equipment by the Pennsylvania Railroad; and the great track exhibit of Mr. A. Haarmann, of Germany.

After the Chicago World's Fair, Mr. Smith temporarily laid aside active management of the *Railway Review* to engage in manufacturing. In 1894, he became President of the Cloud Steel Truck Company of Chicago, and, later, Vice-President of the National Malleable Castings Company.

In 1899, he went to France as Chief of the Department of Transportation and Engineering for the American Commission to the Paris Exposition of 1900. His previous travels in Europe on exposition business, the acquaint-anceships formed, and his eminent success as head of a department at the Chicago Exposition seven years earlier, were experiences that served him to much advantage in his connection with this foreign Exposition. An inno-vation of the American exhibit that had not been anticipated by the French, but which was one of the purposes of Mr. Smith, was the display of American-built steel cars, placed on the grounds with some difficulty by means of barges on the River Seine, of hoisting machinery, and of special tracks. It has been said that this exhibit led to the introduction of steel cars on European rail-ways. On his return from Paris that year he again became actively engaged with the *Railway Review,* continuing as President, which office he had never relinquished while engaged with outside enterprises; and it should not be overlooked that for many years he was also the publisher of the *Railway Master Mechanic* and of the "Railway Official List."

In the organization for the Louisiana Purchase Exposition, in St. Louis, Mo., in 1904, again he was sought as Chief of the Department of Transporta-tion, and again he accepted the office. The exhibit was housed in a single building covering sixteen acres. It included, as in 1893, the historical railway exhibit of the Baltimore and Ohio Railroad, and examples of the heaviest railway rolling stock of the day, locomotives and cars; but perhaps the most worthy effort and result of the exhibition of rolling equipment was the dis-play of the latest refinements of design and service. Notable exhibits of this kind were the numerous designs of modern draft gear; the practical operation and tests of air-brake equipment for an 80-car freight train, by the Westing-house Air Brake Company; and the locomotive testing plant of the Penn-sylvania Railroad, maintained in continuous operation during several weeks. The thoroughness with which this plant was designed and installed within the short time available, and its very satisfactory operation as a testing device, won great praise for the motive power officials of the railroad and for the staff of experts who conducted the tests.

It was through the suggestion and strong appeal of Mr. Smith that the management of the Pennsylvania Railroad had been led to decide on a locomotive testing plant as the distinguishing feature of its exhibit, and he was made a member of the Advisory Board which co-operated with the Rail-road Company in conducting the important series of tests that were there carried out. The other members of this Board were the late Theodore N. Ely, M. Am. Soc. C. E., and J. J. Turner. The data and results of these tests were published in a separate volume by the Railroad Company. The plant was subsequently removed to the Pennsylvania Shops at Altoona, Pa.

Perhaps the most spectacular feature of the Transportation exhibit—at any rate so regarded from the popular standpoint—was a 120-ton, Atlantic type locomotive and tender of the "Big Four" Railway, mounted on a 70-ft. turn-table displayed by the Chicago Bridge and Iron Works. The turn-table was carried on a "center" supported on a concrete pier, in the ordinary way, with the lower flanges of the girders 4 ft. clear of the floor. The locomotive was blocked clear of the turn-table rails, so that its wheels could revolve; and by means of an electric drive both the turn-table and the locomotive drivers were kept continually in motion, the table swinging on its center.

A remarkable feature of the Transportation exhibit, not so well appreciated at that time but now of historic interest, was a Department of Aeronautics, conducted under the immediate direction of the late Octave Chanute, Past-President, Am. Soc. C. E. For this purpose a field of ten acres was fenced off as part of the Exposition Grounds, and gliding experiments were con-ducted at frequent intervals. The motive power was a stationary street-car motor geared to a drum carrying 1 000 ft. of light steel cable. In these experiments the biplanes would often rise to a height of 60 ft. above ground, when the cable would be detached and the pilot would steer the plane around the grounds and make a landing. The attendance of those interested in avia-tion was large, including some of the foremost inventors of France, Italy, Brazil, and other parts of the world, as well as those of the United States, the latter including notably the Wright brothers, who not long after this exhibition achieved epoch-making progress.

At the conclusion of the Exposition, the *Railway Review,* under the direc-tion of Mr. Smith, published a volume covering the railroad exhibits of the Exposition, in much detail. The book contains more than 600 illustrations.

In recognition of Mr. Smith's technical and executive abilities as an organizer of railroad exhibitions, he became the recipient of many honors and decorations. He was appointed and served as a Delegate of the State Depart-ment of the United States Government to the International Railway Congress which met in Paris, in 1900, and in Washington, in 1905. He was honored with medals by the Governments of France, Germany, and Italy, for services in connection with the World's Columbian Exposition; and was presented with the famous Tiffany "Transportation Vase" by the American exhibitors in the Department of Transportation, World's Columbian Exposition, as a testimonial of his eminent services as Chief of that Department. He was decorated Chevalier of the Legion of Honor, by France, and, in 1907, pro-moted to Officer of the Legion of Honor. He was decorated with the Royal Order of the Crown by Germany; and with the Imperial Order of the Rising Sun by Japan. Mr. Smith was the last surviving Chief of Department of the Chicago World's Fair.

He had long been an active member of several railroad and engineering associations, including the Master Car Builders' Association; the American Railway Master Mechanics Association; the Western Society of Engineers; the Western Railway Club; and the American Society of Railroad Super-intendents. Locally, he was a member of the Union League Club, the Chicago

Engineers' Club, and had been a Trustee of the University of Chicago from 1894 until he felt obliged, by failing health, to tender his resignation, a few months before his death. For a long time he served as the Chairman of the Press and Extension Committee of that institution. He was a Patron and Honorary Curator of the Field Museum of Natural History and, for many years, a Trustee of the Memorial Baptist Church. He also served as President of the Chicago Baptist Social Union for two years.

Mr. Smith was married on May 1, 1872, to Maria C. Dickinson, of St. Louis, Mo., who passed away suddenly about five years before his death. He is survived by three children: Mrs. B. V. Crandall, of Kenilworth, Ill.; and Miss Edith and Harold A. Smith, of Highland Park, Ill., the last named becoming President of the *Railway Review* on the death of his father.

In his private and social life, as well as in his business career, Mr. Smith was widely recognized as a fine type of gentleman. He was a man of extraordinary intellectual force and culture, with broad and charitable views; and to know him was to admire him. His impulses were humanitarian, and the serious bent of his nature was educational. He had been a student all his life, and a constant reader of the best literary talent of his time.

Schooled by contact with some of the best engineering minds and methods of the country, and having intimate knowledge of the progress of the design of railway rolling stock, shop, and other equipment, he was not infrequently called into consultation for opinions or advice on engineering enterprises.

Mr. Smith was elected an Affiliate of the American Society of Civil Engineers on September 2, 1914.